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The purpose of these standards is to establish minimum engineering, inspection and maintenance criteria for marine oil terminals, to prevent oil spills and to protect the public health, safety and the environment, as well as the infrastructure of the State of California.

MARINE OIL TERMINAL ENGINEERING & MAINTENANCE STANDARDS

CALIFORNIA STATE LANDS COMMISSION

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1. INTRODUCTION

1.1 GENERAL

The Lempert-Keene-Seastrand Oil Spill Prevention and Response Act of 1990 (Act), as amended, authorized the California State Lands Commission (CSLC) to regulate (Public Resources Code Sections 8755 and 8756) Marine Oil Terminals (MOTs) in order to protect public health, safety, and the environment by preventing oil spills. The Marine Facilities Division (Division) of CSLC was created to carry out the mandates of the Act. This Act defines "oil" as any kind of petroleum, liquid hydrocarbons, or petroleum products or any fraction or residues therefrom, including but not limited to, crude oil, bunker fuel, gasoline, diesel fuel, aviation fuel, oil sludge, oil refuse, oil mixed with waste, and liquid distillates from unprocessed natural gas.

These Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS) have been developed by the CSLC with additional funding from the Federal Emergency Management Agency (FEMA), through a hazard mitigation grant provided through the California Governor's Office of Emergency Services.

1.2 PURPOSE

The purpose of these standards is to establish minimum engineering, inspection and maintenance criteria for MOTs in order to prevent oil spills and to protect public health, safety and the environment. The standards do not, in general, address operational requirements. Appropriate existing codes, industry standards, recommended practices and guidelines have been incorporated as part of these standards.

1.3 APPLICABILITY

The provisions of these standards apply to all new and existing MOTs in California.

Any MOT or berthing system that commences operation after adoption of these standards shall be considered "new".

The addition of new structural components or systems on existing MOTs that are structurally independent of the existing components or systems, shall conform to the "new" requirements of Sections 3 through 7.

The terms "new" and "existing", and corresponding requirements specific to Sections 8 through 11 are defined in each individual Section.

1.4 OVERVIEW

These standards will ensure that a MOT can be safely operated within its inherent structural and equipment-related constraints. The standards include inspection procedures of all structural, electrical and mechanical systems on a prescribed periodic basis, or following a significant damage-causing event, in order to verify that each berthing system is fit for its specific, defined purpose. Figure 1-1 provides a flowchart for the implementation of these standards at existing MOTs.

Section 2 defines minimum requirements for auditing, inspecting and evaluating the structural, electrical and mechanical systems at MOTs.

Sections 3, 4 and 7 provide criteria for structural loading, deformation and performance-based evaluation including the use of site-specific data for earthquake, wind, wave, current, seiche and tsunami effects at MOTs. The structural requirements for seismic evaluation presented herein follow, to a large degree, the philosophy presented in FEMA-356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings."

Section 5 provides requirements for MOTs, regarding safe mooring and berthing of tank vessels and barges.

Section 6 provides requirements for seismic hazards and foundation analyses, including consideration of slope stability and soil failure mechanisms.

Section 8 provides requirements for a fire plan, including appropriate water and foam volumes considering reasonable worst case fire and explosion scenarios.

Sections 9 through 11 provide requirements for piping, electrical and mechanical equipment.

English units are prescribed herein; however, many of the units in the references are in System International (SI).

1.5 RISK REDUCTION STRATEGIES

Risk reduction strategies, such as pipeline segmentation devices, system flexibility and spill containment devices may be used to reduce the size of a potential oil spill. Such strategies may reduce the MOT risk category as determined in Table 4-1. Strategies may be selected that exceed the requirements of these MOTEMS for long-term cost-benefits.

1.6 REVIEW REQUIREMENTS

1.6.1 Quality Assurance

All engineering analyses performed in accordance with these standards shall be reviewed to ensure quality assurance. Quality assurance may be performed in-house.

1.6.2 Structural Analysis Peer Review

Peer review is required for nonlinear dynamic structural analyses and alternative lateral force procedures not prescribed in these Standards. The peer review may be from an independent internal or external source. The peer reviewer shall be a California registered civil or structural engineer.

1.6.3 Division Review

Any analysis or design for new or existing MOTs, prescribed by these standards is subject to review and approval by the Division or their designated representatives.

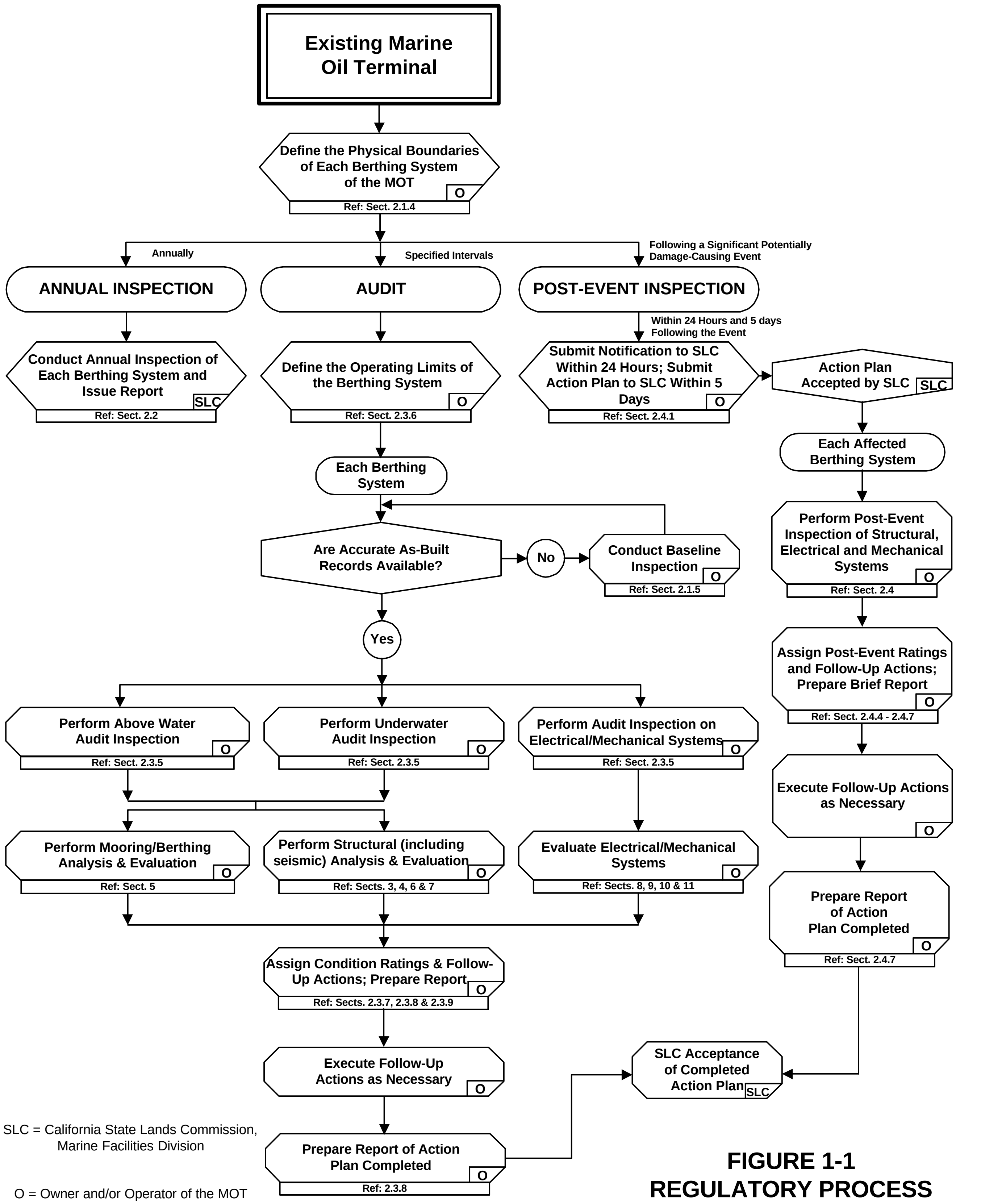

FIGURE 1-1
REGULATORY PROCESS FLOWCHART

2. AUDIT AND INSPECTION

2.1 GENERAL

2.1.1 Purpose

Section 2 defines minimum requirements for auditing, inspecting, and evaluating the structural, electrical and mechanical components and systems at MOTs.

2.1.2 Applicable Codes and Standards

Childs, K.M., editor, 2001, **"**Underwater Investigations - Standard Practice Manual," American Society of Civil Engineers, Reston, VA.

California Code of Regulations (CCR), Title 2, Division 3, Chapter 1, Article 5, Marine Terminals Inspection and Monitoring, Sections 2315, 2320, 2325, and 2385 (short form example: 2 CCR 2315 = Title 2 of California Code of Regulations, Section 2315).

2.1.3 Types of Audits and Inspections

Audits and inspections required under these standards and 2 CCR 2320 (a)(1) are:

- Annual Inspection
- Audit
- Post-Event Inspection

Each has a distinct purpose and is conducted either at a defined interval (See Tables 2-1 and 2-2), as a result of a potentially damaging event or a significant change in operations. Between audits, operators are expected to conduct periodic walk-down inspections of the MOT to detect potentially unsafe conditions.

2.1.4 Physical Boundaries

The physical boundaries comprising a MOT shall be divided into independent "berthing systems" for assigning structural ratings and documenting electrical/mechanical deficiencies. A berthing system is defined as the complete set of structural, electrical and mechanical components for the transfer of product to or from a vessel to the shoreside valve closest to the onshore containment area onshore.

For example, a MOT consisting of wharves with three berths adjacent to the shoreline could contain three independent "berthing systems" if the piping does not route through adjacent berths. Therefore, a significant defect that would restrict the operation of one berth would have no impact on the other two berths. Conversely, if a T-head Pier (with multiple berths sharing a trestle which supports all piping to the shoreline) had a significant deficiency on the common trestle, the operation of all berths could be adversely impacted. This configuration is classified as a single berthing system.

The physical boundaries of a berthing system may exclude unused sections of a structure. Excluded sections must be physically isolated from the berthing system. Expansion joints may provide this isolation.

2.1.5 Records

A. Design Basis Records

All MOTs shall maintain the records of modifications and new construction including design drawings, calculations, engineering analyses, soil borings, equipment manuals, specifications, shop drawings, technical and maintenance manuals and documents. All of this information shall reflect current, as-built conditions.

Records shall be indexed and be readily accessible to the Division (2 CCR Section 2320 (c) (2)).

B. Baseline Inspection

If "as-built" or subsequent modification drawings showing structural, electrical and mechanical systems information are not available, incomplete, or inaccurate, the Audit must include a Baseline Inspection to gather data in sufficient detail to adequately evaluate the MOT.

The level of detail required shall be such that structural member sizes, connection and reinforcing details are documented, if required in the structural analysis. In addition, the strength and/or ductility characteristics of construction materials shall be determined, as appropriate. This may require the use of non-destructive testing, partially destructive testing and/or laboratory testing methods.

All fire, piping, electrical and mechanical systems shall be documented as to location, capacity, operating limits, and physical conditions.

C. Modifications and Replacements

Where modifications and/or replacement of structural components, electrical or mechanical equipment or relevant operational changes are made, the records shall be updated.

D. Audit and Inspection Records

Chronological records and reports of Annual Inspections, Audits and Post-Event Inspections and documentation of equipment or structural changes shall be maintained.

All of these records and reports shall be readily accessible to the Division (2 CCR Section 2320 (c)(2)).

2.2 ANNUAL INSPECTION

The Annual Inspection required by 2 CCR 2320 (a)(1), may include an engineering examination of the topside and underside areas of the dock, including the splash zone. The Division shall perform the inspection, with cooperation from the owner/operator. Observations will be recorded and a report of violations and deficiencies shall be provided to the operator.

Subject to MOT operating procedures, a boat shall be provided to facilitate the inspection of the dock undersides and piles to the splash zone. If a boat is not available or the under dock inspection cannot be performed during the Annual Inspection, the MOT shall provide the Division with inspection results including photographs, videos or sketches of deficiencies, from the splash zone to the dock undersides.

2.3 AUDIT

2.3.1 Objective

The objective of the Audit is to review structural, electrical and mechanical systems on a prescribed periodic basis to verify that each berthing system is fit for its specific defined purpose. The Audit includes both above water and underwater inspections, as well as engineering analyses, as necessary, to confirm the fitness of the MOT for the defined purpose.

2.3.2 Overview

The Audit provides an assessment of structural, electrical, mechanical and mooring systems, relative to their fitness-for-purpose. The Initial Audit shall include above water and underwater structural inspection, mooring, berthing and structural evaluation, and electrical/mechanical systems evaluation. The audit is performed by a multi-disciplinary team of engineers, qualified inspectors and may also include Division representatives.

The above water inspection involves an examination of all accessible structural, electrical and mechanical components above the waterline. Structural defects and their severity shall be documented, but the exact size and location of each deficiency is typically not required.

Since underwater visibility is limited, portions of components are typically covered with marine growth, and inspection time may be restricted by environmental factors (currents, water temperatures, etc.), a visual inspection of all components during an underwater inspection may not be feasible. In such circumstances, the inspection may be limited to representative sampling [2.1].

A Condition Assessment Rating (CAR) shall be assigned to above and underwater structural systems. Deficiencies and Remedial Action Priorities (RAP) shall be assigned for electrical and mechanical systems. Recommendations for remediation and/or upgrading shall be prescribed as necessary. The CAR and RAP will remain in effect for each system until such time as repairs or upgrades are made.

Subsequent audits of the above water and underwater structures and electrical/mechanical systems may or may not be performed concurrently, depending upon the required inspection intervals based on the prior audit report.

2.3.3 Schedule

A. Initial Audit

Table 2-1 provides the deadlines for the submission of the Initial Audit report. MOT classification in Table 2-1 is determined from the higher assigned risk category obtained from Tables 4-1 and 5-1.

For an entirely new MOT berthing system, the Initial Audit shall be performed within three years of commencement of operations.

TABLE 2-1 INITIAL AUDIT REPORT SUBMISSION DEADLINE FOR EXISTING BERTHING SYSTEMS	
MOT Classification[1]	**Submission Deadline[2]**
High	30 Months
Medium	48 Months
Low	60 Months

1 As defined in Sections 4 and 5, Tables 4-1 and 5-1
2 From the effective date of this standard

B. Subsequent Audits

The above water Audit of structural, electrical and mechanical systems, including both the inspection and evaluation of these systems as defined in Section 2.3.5 and 2.3.6 shall be completed at a maximum interval of 3 years. This interval may be reduced, based on the recommendation of the Audit Team Leader, and with the approval of the Division, depending on the extent and rate of deterioration or other factors.

The maximum interval for underwater Audits is dependent upon the condition of the facility and the construction material type, as shown in Table 2-2.

If there are no changes in the defined purpose of the berthing system, then analyses from previous Audits may be referenced. However, if there is a significant change in a berthing system, or when deterioration or damage must be considered, a new analysis may be required.

The Audit is not considered complete until the Audit Report is received by the Division.

C. Change in Operations

The Division may require an Audit to justify changes in use of a berthing system. Examples of such changes include berthing and mooring of larger or smaller vessels considering dolphin and fender spacing, or more stringent environmental conditions. These Audits are limited to berthing systems or components affected by the change.

2.3.4 Audit Team

A. Project Manager

The Audit shall be conducted by a multi-disciplinary team under the direction of a Project Manager representing the MOT. The Project Manager shall have specific knowledge of the MOT and may serve other roles on the Audit Team.

TABLE 2-2

MAXIMUM INTERVAL BETWEEN UNDERWATER AUDIT INSPECTIONS (YEARS)[1]

Condition Rating From Previous Inspection	CONSTRUCTION MATERIAL: Unwrapped Timber or Unprotected Steel (no coating or cathodic protection)[4] – Benign[2] Environment	CONSTRUCTION MATERIAL: Unwrapped Timber or Unprotected Steel (no coating or cathodic protection)[4] – Aggressive[3] Environment	CONSTRUCTION MATERIAL: Concrete, Wrapped Timber, Protected Steel or Composite Materials (FRP, plastic, etc.)[4] – Benign[2] Environment	CONSTRUCTION MATERIAL: Concrete, Wrapped Timber, Protected Steel or Composite Materials (FRP, plastic, etc.)[4] – Aggressive[3] Environment	Channel Bottom or Mudline – Scour[4] – Benign[2] Environment	Channel Bottom or Mudline – Scour[4] – Aggressive[3] Environment
6 (Good)	6	4	6	5	6	5
5 (Satisfactory)	6	4	6	5	6	5
4 (Fair)	5	3	5	4	6	5
3 (Poor)	4	3	5	4	6	5
2 (Serious)	2	1	2	2	2	2
1 (Critical)	N/A[5]	N/A[5]	N/A[5]	N/A[5]	N/A[5]	N/A[5]

1. The maximum interval between Underwater Audit Inspections shall be reduced as appropriate based on the extent of deterioration observed on a structure, the rate of further anticipated deterioration, or other factors.
2. Benign environments include fresh water and maximum current velocities less than 1.5 knots for the majority of the days in a calendar year
3. Aggressive environments include brackish or salt water, polluted water, or waters with current velocities greater than 1.5 knots for the majority of the days in the calendar year.
4. For most structures, two maximum intervals will be shown in this table, one for the assessment of construction material (timber, concrete, steel, etc) and one for scour (last 2 columns). The shorter interval of the two should dictate the maximum interval used.
5. MOTs rated “Critical” will not be operational; and Emergency Action shall be required in accordance with Table 2-7.

B. Audit Team Leader

The Audit Team Leader shall lead the on-site audit team and shall be responsible for directing field activities, including the inspection of all structural, electrical and mechanical systems. The Team Leader shall be a California registered civil or structural engineer and may serve other roles on the audit team.

C. Structural Inspection Team

The structural inspection shall be conducted under the direction of a registered civil or structural engineer. For underwater inspections, the registered civil or structural engineer directing the underwater structural inspection shall also be a commercially trained diver or equivalent and shall actively participate in the inspection, by personally conducting a minimum of 25 percent of the underwater examination.

All members of the structural inspection team shall be graduates of a 4-year civil/structural engineering, or closely related (ocean/coastal) engineering curriculum, and shall have been certified as an Engineer-in-Training; or shall be technicians who have completed a course of study in structural inspections. The minimum acceptable course in structural inspections shall include 80 hours of instruction specifically related to structural inspection, followed by successful completion of a comprehensive examination. An example of an acceptable course is the U.S. Department of Transportation's "Safety Inspection of In-Service Bridges". Certification as a Level IV Bridge Inspector by the National Institute of Certification in Engineering Technologies (NICET) shall also be acceptable.

For underwater inspections, each underwater team member shall also be a commercially trained diver, or equivalent. Divers performing manual tasks, such as cleaning or supporting the diving operation, but not conducting or reporting on inspections may have lesser technical qualifications.

D. Seismic Structural Analyst

A California registered civil or structural engineer shall perform the seismic structural evaluation required for the Audit.

E. Electrical Inspection Team

A registered electrical engineer shall direct the on-site team performing the inspection and evaluation of electrical components and systems.

F. Mechanical Inspection Team

A registered engineer shall direct the on-site team performing the inspection of pipeline, mechanical and fire systems.

G. Division Representation

The Division representative(s) may participate in any Audit as observer(s) and may provide guidance.

2.3.5 Scope of Inspection

A. Above Water Structural Inspection

The above water inspection shall include all accessible components above +3 ft MLLW. Accessible components shall be defined as those components above and below deck that are accessible without the need for excavation or extensive removal of materials that may impair visual inspection. The above water inspection shall include but not limited to the following:

- Piles
- Pile caps
- Beams
- Deck soffit
- Bracing
- Retaining walls and Bulkheads
- Connections
- Seawalls
- Slope protection
- Deck topsides and curbing
- Expansion joints
- Fender system components
- Dolphins and deadmen
- Mooring points and hardware
- Navigation aids
- Platforms, ladders, stairs, handrails and gangways
- Backfill (sinkholes/differential settlement)

B. Underwater Structural Inspection

The underwater inspection shall include all accessible components from +3 ft MLLW to the mudline, including the slope and slope protection, in areas immediately surrounding the MOT. The water depth at the berth(s) shall be evaluated, verifying the maximum or loaded draft specified in the MOT's Operations Manual (2 CCR 2385 (d)).

The underwater structural inspection shall include the Level I, II, and III inspection efforts, as shown in Tables 2-3 and 2-4. The underwater inspection levels of effort are described below:

(1) Level I Effort – Includes a close visual examination, or a tactile examination using large sweeping motions of the hands where visibility is limited. Although the Level I effort is often referred to as a "Swim-By" inspection, it must be detailed enough to detect obvious major damage or deterioration due to overstress or other severe deterioration. It should confirm the continuity of the full length of all members and detect undermining or exposure of normally buried elements. A Level I effort may also include limited probing of the substructure and adjacent channel bottom.

(2) Level II Effort – A detailed inspection which requires marine growth removal from a representative sampling of components within the structure. For piles, a 12-inch high band should be cleaned at designated locations, generally near the low waterline, at the mudline, and midway between the low waterline and the mudline.

On a rectangular pile, the marine growth removal should include at least three sides; on an octagon pile, at least six sides; on a round pile, at least three-fourths of the perimeter. On large diameter piles, 3 ft or greater, marine growth removal should be effected on 1 ft by 1ft areas at four locations approximately equally spaced around the perimeter, at each elevation. On large solid faced elements such as retaining structures, marine growth removal should be effected on 1 ft by 1 ft areas at the three specified elevations. The inspection should also focus on typical areas of weakness, such as attachment points and welds. The Level II effort is intended to detect and identify damaged and deteriorated areas that may be hidden by surface biofouling. The thoroughness of marine growth removal should be governed by what is necessary to discern the condition of the underlying structural material. Removal of all biofouling staining is generally not required.

(3) Level III Effort – A detailed inspection typically involving non-destructive or partially-destructive testing, conducted to detect hidden or interior damage, or to evaluate material homogeneity.

TABLE 2-3
UNDERWATER INSPECTION LEVELS OF EFFORT

Level	Purpose	Detectable Defects			
		Steel	**Concrete**	**Timber**	**Composite**
I	General visual/tactile inspection to confirm as-built condition and detect severe damage	Extensive corrosion, holes Severe mechanical damage	Major spalling and cracking Severe reinforcement corrosion Broken piles	Major loss of section Broken piles and bracings Severe abrasion or marine borer attack	Permanent deformation Broken piles Major cracking or mechanical damage
II	To detect surface defects normally obscured by marine growth	Moderate mechanical damage Corrosion pitting and loss of section	Surface cracking and spalling Rust staining Exposed reinforcing steel and/or prestressing strands	External pile damage due to marine borers Splintered piles Loss of bolts and fasteners Rot or insect infestation	Cracking Delamination Material degradation
III	To detect hidden or interior damage, evaluate loss of cross-sectional area, or evaluate material homogeneity	Thickness of material Electrical potentials for cathodic protection	Location of reinforcing steel Beginning of corrosion of reinforcing steel Internal voids Change in material strength	Internal damage due to marine borers (internal voids) Decrease in material strength	N/A

TABLE 2-4
SCOPE OF UNDERWATER INSPECTIONS

Level		Sample Size and Methodology[1, 2]							
		Steel		Concrete		Timber		Composite	Slope Protection/ Channel Bottom or Mudline-Scour
		Piles	Bulkheads/ Retaining Walls	Piles	Bulkheads/ Retaining Walls	Piles	Bulkheads/ Retaining Walls	Piles	
I	Sample Size:	100%	100%	100%	100%	100%	100%	100%	100%
	Method:	Visual/ Tactile	Visual/ Tactile	Visual/ Tactile	Visual/ Tactile	Visual/ Tactile	Visual/ Tactile	Visual/ Tactile	Visual/ Tactile
II	Sample Size:	10%	Every 100 LF	10%	Every 100 LF	10%	Every 50 LF	10%	0%
	Method:	Visual: Removal of marine growth in 3 bands	Visual: Removal of marine growth in 1 SF areas	Visual: Removal of marine growth in 3 bands	Visual: Removal of marine growth in 1 SF areas	Visual: Removal of marine growth on 3 bands Measurement: Remaining diameter	Visual: Removal of marine growth in 1 SF areas	Visual: Removal of marine growth in 3 bands	
III	Sample Size:	5%	Every 200 LF	0%	0%	5%	Every 100 LF	0%	0%
	Method:	Remaining thickness measurement; electrical potential measurement; corrosion profiling as necessary	Remaining thickness measurement; electrical potential measurement; corrosion profiling as necessary	N/A	N/A	Internal marine borer infestation evaluation	Internal marine borer infestation evaluation		

1. The stated sample size may be reduced in the case of large structures where statistically representative sampling can be demonstrated to the Division in accordance with these standards. The sampling plan must be representative of all areas and component types (i.e. approach trestles, pier/wharf, dolphins, inboard, outboard, batter, vertical, concrete, steel, timber, etc.). Any reduced sampling plan proposed to the Division must include the Level I inspection of all piles around the perimeter of the facility where vessels may berth or where debris may impact or accumulate. If the reduced sampling plan proposes to conduct less than 100 percent Level I effort, then the results of the inspection must be carefully monitored. If significant deterioration is observed on any component, which could reasonably be expected to be present on additional components, and which could have a detrimental effect on the load bearing capacity of the structure either locally or globally, then the inspection scope shall be increased to include a 100 percent Level I effort. See reference [2.1].
2. The minimum inspection sampling size for small structures shall include at least two components.

LF = Linear Feet; SF = Square Feet, N/A = Not Applicable

Typical inspection and testing techniques include the use of ultrasonics, coring or boring, physical material sampling and in-situ hardness testing. Level III testing is generally limited to key structural areas, areas which are suspect, or areas which may be representative of the underwater structure.

C. Special Inspection Considerations

(1) Coated Components - For coated steel components, Level I and II efforts should focus on the evaluation of the integrity and effectiveness of the coating. The piles should be inspected without damaging the coating. Level III efforts should include ultrasonic thickness measurements without removal of the coating, where feasible.

(2) Encased Components - For steel, concrete or timber components which have been encased, the Level I and II efforts should focus on the evaluation of the integrity of the encasement. If evidence of significant damage to the encasement is present, or if evidence of significant deterioration of the underlying component is present, then the damage evaluation should consider whether the encasement was provided for protection and/or structural capacity. Encasements should not typically be removed for an Audit.

For encasements on which the formwork has been left in place, the inspection should focus on the integrity of the encasement, not the formwork. Level I and II efforts in such cases should concentrate on the top and bottom of the encasement. For concrete components, if deterioration, loss of bonding, or other significant problems with the encasement are suspected, it may be necessary to conduct a Special Inspection, including coring of the encasement and laboratory evaluation of the materials.

(3) Wrapped Components - For steel, concrete or timber components that have been wrapped, the Level I and II efforts should focus on the evaluation of the integrity of the wrap. Since the effectiveness of a wrap may be compromised by removal, and since the removal and re-installation of wraps is time-consuming, it should not be routinely done. However, if evidence of significant damage exists, or if the effectiveness of the wrap may be in question, then a sample of wraps should be removed to facilitate the inspection and evaluation. The sample may be limited to particular zones or portions of the member if damage is suspected, for example, at the waterline or mudline. The sample size should be determined based on the physical evidence of potential problems. A minimum sample size of three members should be used. A five-percent sample size, up to 30 total members, may be adequate as an upper limit.

For timber components, Level III efforts should consist of removal of the wraps from a representative sample of components in order to evaluate the condition of the timber beneath the wrap. The sample may be limited to particular zones or portions of the member if damage is suspected; e.g. at the mudline/bottom of wrap or in the tidal zone. The sample size should be determined based on the physical evidence of potential problems and the aggressiveness of the environment. A minimum sample size of three members should be used. A five-percent sample size, up to 30 total members, may be adequate as an upper limit.

D. Electrical and Mechanical Equipment

The inspection of electrical and mechanical equipment shall include but not be limited to the following components and systems:

- Loading arms
- Cranes and lifting equipment, including cables
- Piping/manifolds and supports
- Oil transfer hoses
- Fire detection and suppression systems
- Vapor control system
- Sumps/sump tanks
- Vent systems
- Pumps and pump systems
- Lighting
- Communications equipment
- Gangways
- Electrical switches and junction boxes
- Emergency power equipment
- Air compressors
- Meters
- Cathodic protection systems
- Winches
- ESD and other control systems
- Ladders

All alarms, limit switches, load cells, current meters, anemometers, leak detection equipment, etc., shall be operated/tested and calibrated, to the extent feasible, to ensure proper function.

2.3.6 Evaluation and Assessment

A. Terminal Operating Limits

The physical boundaries of the facility shall be defined by the berthing system operating limits, along with the vessel size limits and environmental conditions.

The Audit shall include a “Statement of Terminal Operating Limits”, which must provide a concise statement of the purpose of each berthing system in terms of operating limits. This description must at least include, the minimum and maximum vessel sizes, including LOA, beam, and maximum draft with associated displacement. A typical example is provided in Figure 2-1.

In establishing limits for both the minimum and maximum vessel sizes, due consideration shall be given to water depths, dolphin spacing, fender system limitations, manifold height and hose/loading arm reach, with allowances for tidal fluctuations, surge, and drift.

Maximum wind, current, or wave conditions, or combinations thereof, shall be clearly defined as limiting conditions for vessels at each berth, both with and without active product transfer.

B. Mooring and Berthing

Mooring and berthing analyses shall be performed in accordance with Section 5. The analyses shall be consistent with the terminal operating limits and the structural configuration of the wharf and/or dolphins and associated hardware. The results and supporting documentation shall be provided.

C. Structure

A structural evaluation, including a seismic analysis, shall be performed in accordance with Sections 3 through 7. Such evaluations shall consider local or global reduction in capacity, as determined from the inspection.

Based on inspection results, structural analyses and engineering judgment, separate CARs shall be assigned independently for above and underwater structures. The CAR documents the evaluation of structural fitness-for-purpose. The CARs defined in Table 2-5 shall be used for this purpose. The assigned ratings shall remain in effect until the MOT has completed all significant corrective action to the satisfaction of the Division, or until completion of the next Audit.

D. Electrical and Mechanical Systems

An evaluation of all electrical and mechanical systems and components shall be performed in accordance with Sections 8 through 11 of this standard. If a pipeline analysis is required, it shall consider forces and imposed seismic displacements resulting from the structural analysis. Table 2-6 presents the RAP for electrical and mechanical systems. The results and supporting documentation of these evaluations shall be provided.

2.3.7 Follow-up Actions

Structural follow-up actions as described in Table 2-7 shall be assigned. Multiple follow-up actions may be assigned; however, guidance should be provided as to the order in which the follow-up actions should be carried out. Table 2-6 presents the remedial priorities and actions for electrical and mechanical systems. If the remedial measure is “Priority 1” (Table 2-6) or “Emergency” (Table 2-7), the Division shall be notified immediately. The audit report shall include implementation schedules for all follow-up and remedial actions. Follow-up and remedial actions and implementation schedules are subject to Division approval. Follow-up actions shall also state the maximum interval to the next audit.

2.3.8 Documentation and Reporting

The audit report shall be signed and stamped by the Audit Team Leader.

Each Audit, whether partial or complete, shall be adequately documented. Partial audits cover only specific systems or equipment examined. The resulting report shall summarize and reference relevant previous ratings and deficiencies.

The contents of the audit report for each berthing system shall, at a minimum, include the following as appropriate:

Executive Summary – a concise summary of the audit results and analyses conclusions. It shall include summary information for each berthing system, including an overview of the assigned follow-up actions (See Example Tables ES-1 and ES-2).

Table of Contents

Body of Report

Introduction – a brief description of the purpose and scope of the audit, as well as a description of the inspection/evaluation methodology used for the audit.

Existing Conditions – a brief description of the MOT, along with a summary of the observed conditions. Subsections should be used to describe the above water structure, underwater structure and electrical/mechanical systems, to the extent each are included in the scope of the audit. Photos, plan views and sketches shall be utilized as appropriate to describe the structure and the observed conditions. Details of the inspection results such as test data, measurements data, etc. shall be documented in an appendix.

Evaluation and Assessment - a CAR shall be assigned to structural systems (above and under water). Mooring and berthing analyses, structural analysis results, and all supporting calculations shall be included in appendices as appropriate to substantiate the ratings. However, the results and recommendations of the engineering analyses shall be included in this section. Electrical and

TABLE 2-5

CONDITION ASSESSMENT RATINGS (CAR)

Rating		Description of Structural Systems Above and Below Water Line
6	**Good**	No problems or only minor problems noted. Structural elements may show very minor deterioration, but no overstressing observed. The capacity of the structure meets the requirements of this standard. The structure should be considered fit-for-purpose. No repairs or upgrades are required.
5	**Satisfactory**	Limited minor to moderate defects or deterioration observed, but no overstressing observed. The capacity of the structure meets the requirements of this standard. The structure should be considered fit-for-purpose. No repairs or upgrades are required.
4	**Fair**	All primary structural elements are sound; but minor to moderate defects or deterioration observed. Localized areas of moderate to advanced deterioration may be present, but do not significantly reduce the load bearing capacity of the structure. The capacity of the structure is no more than 15 percent below the structural requirements of this standard, as determined from an engineering evaluation. The structure should be considered as marginal. Repair and/or upgrade measures may be required to remain operational. Facility may remain operational provided a plan and schedule for remedial action is presented to and accepted by the Division.
3	**Poor**	Advanced deterioration or overstressing observed on widespread portions of the structure, but does not significantly reduce the load bearing capacity of the structure. The capacity of the structure is no more than 25 percent below the structural requirements of this standard, as determined from an engineering evaluation. The structure is not fit-for-purpose. Repair and/or upgrade measures may be required to remain operational. The facility may be allowed to remain operational on a restricted or contingency basis until the deficiencies are corrected, provided a plan and schedule for such work is presented to and accepted by the Division.
2	**Serious**	Advanced deterioration, overstressing or breakage may have significantly affected the load bearing capacity of primary structural components. Local failures are possible and loading restrictions may be necessary. The capacity of the structure is more than 25 percent below than the structural requirements of this standard, as determined from an engineering evaluation. The structure is not fit-for-purpose. Repairs and/or upgrade measures may be required to remain operational. The facility may be allowed to remain operational on a restricted basis until the deficiencies are corrected, provided a plan and schedule for such work is presented to and accepted by the Division.
1	**Critical**	Very advanced deterioration, overstressing or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur and load restrictions should be implemented as necessary. The capacity of the structure is critically deficient relative to the structural requirements of this standard. The structure is not fit-for-purpose. The facility shall cease operations until deficiencies are corrected and accepted by the Division.

TABLE 2-6

REMEDIAL ACTION PRIORITIES (RAP) FOR ELECTRICAL AND MECHANICAL DEFICIENCIES

Remedial Priorities	Description and Remedial Actions
P1	Specified whenever a condition that poses an immediate threat to public health, safety or the environment is observed. <u>Emergency Actions</u> may consist of barricading or closing all or portions of the berthing system, evacuating product lines and ceasing transfer operations. The berthing system is not fit-for-purpose. <u>Immediate remedial actions are required prior to the continuance of normal operations.</u>
P2	Specified whenever defects or deficiencies pose a potential threat to public health, safety and the environment. Actions may consist of limiting or restricting operations until remedial measures have been completed. The berthing system is not fit-for-purpose. This priority requires investigation, evaluation and <u>urgent action.</u>
P3	Specified whenever systems require upgrading in order to comply with the requirement of these standards or current applicable codes. These deficiencies <u>do not require emergency or urgent actions.</u> The berthing system is not fit-for-purpose. The MOT may have limitations placed on its operational status.
P4	Specified whenever damage or defects requiring repair are observed. The berthing system is fit-for-purpose. <u>Repair can be performed during normal maintenance cycles, but not to exceed one year.</u>

TABLE 2-7

STRUCTURAL FOLLOW-UP ACTIONS

Follow-up Action	Description
Emergency Action	Specified whenever a condition which poses an immediate threat to public health, safety or the environment is observed. Emergency Actions may consist of barricading or closing all or portions of the berthing system, limiting vessel size, placing load restrictions, evacuating product lines, ceasing transfer operations, etc.
Engineering Evaluation	Specified whenever structural damage or deficiencies are observed which require further investigation or evaluation, to determine appropriate follow-up actions.
Repair Design Inspection	Specified whenever damage or defects requiring repair are observed. The repair design inspection is performed to the level of detail necessary to prepare appropriate repair plans, specifications and estimates.
Upgrade Design and Implementation	Specified whenever the structural system requires upgrading in order to comply with the requirements of these standards and current applicable codes.
Special Inspection	Typically specified to determine the cause or significance of non-typical deterioration, usually prior to designing repairs. Special testing, laboratory analysis, monitoring or investigation using non-standard equipment or techniques are typically required.
Develop and Implement Repair Plans	Specified when the Repair Design Inspection and required Special Inspections have been completed. Indicates that the structure is ready to have repair plans prepared and implemented.
No Action	Specified when no further action is necessary until the next scheduled audit or inspection.

mechanical deficiencies should be described and a RAP assigned to each.

Follow-up Actions – Specific follow-up actions shall be documented (Table 2-7) and remedial schedules included, for each audited system. Audit Team Leaders shall specify which follow-up actions require a California registered engineer to certify that the completion is acceptable.

Appendices – When appropriate, the following appendices shall be included:

- Background data on the terminal - description of the service environment (wind/waves/ currents), extent and type of marine growth, unusual environmental conditions, etc.
- Inspection/Testing Data
- Mooring and Berthing Analyses
- Structural and Seismic Analyses and Calculations
- Fire Plan
- Pipeline stress and displacement analyses
- Electrical and Mechanical System Analysis and Calculations
- Photographs and/or sketches shall be included to document typical conditions and referenced deficiencies, and to justify CARs and RAPs.
- Condition Assessment Rating (CAR) – summary of the rating for each structural system (Table 2-5)
- Remedial Action Priorities (RAP) – summary of the remedial priorities for electrical and mechanical deficiencies (Table 2-6)

2.3.9 Action Plan Implementation Report

Within 90 days of completion of the remedial measures (for serious deficiencies, such as P1, P2, or any structural CAR less than 5) specified in the follow-up action plan(s), a report shall be submitted to the Division and shall include:

- A description of each action taken
- Updated CARs assigned to the structural systems (above and under water)
- Supporting documentation with calculations and/or relevant data

Example	EXECUTIVE SUMMARY TABLE (ES-1) STRUCTURAL CONDITION ASSESSMENT RATINGS (CAR)						
Berthing System	**System**	**Condition Assessment Rating**	**From this Audit[1]**	**From Previous Audit[1]**	**Next Audit Due (Mo/Yr)**	**Assigned Follow-Up Actions**	**Fit-for-Purpose?**
North Wharf	Above Water Structure	4 (Fair)	✓(date)		6/2004	Upgrade Design and Implementation	No
	Underwater Structure	5 (Satisfactory)		✓(date)	10/2006		Yes
South Wharf	Above Water Structure	4 (Fair)	✓(date)		6/2004	Repair Design Inspection	No
	Underwater Structure	3 (Poor)		✓(date)	10/2006	Special Inspection; Repair Design Inspection	No
Dolphin, Trestle, etc.							

1. Place check mark and date of respective audit in proper column to indicate for each structural system, whether the system was included in the current audit or the results are summarized from a previous audit.

Example	EXECUTIVE SUMMARY TABLE (ES-2) ELECTRICAL AND MECHANICAL SYSTEM REMEDIAL ACTION PRIORITIES (RAP)						
Berthing System	**Deficiency**	**Remedial Action Priority (RAP) (P1-P4)**	**From this Audit**	**From Previous Audit**	**Next Audit Due (Mo/Yr)**	**Description of Planned Remedial Action**	**Fit-For-Purpose?**
North Wharf	Fire main leaking	**P3**		✓(date)	6/2004	Repair	No
	Pipeline badly corroded	**P2**	✓(date)			Investigate; urgent action required	
	Electrical (Class I, Div 2 violation)	**P1**	✓(date)			Immediate remedial action required	

2.4 POST-EVENT INSPECTION

A Post-Event Inspection is a focused inspection following a significant, potentially damage-causing event such as an earthquake, storm, vessel impact, fire, explosion or tsunami. The primary purpose is to assess the integrity of structural, electrical and mechanical systems. This assessment will determine the operational status and/or any remedial measures required.

2.4.1 Notification and Action Plan

Notification as per 2 CCR 2325(e) shall be provided to the local area Division field office. The notification shall include, as a minimum:

- Brief description of the event
- Brief description of the nature, extent and significance of any damage observed as a result of the event
- Operational status and any required restrictions.
- Statement as to whether a Post-Event Inspection will be carried-out

The Division may carry out or cause to be carried out, a Post-Event Inspection. In the interim, the Division may direct a change in the Operations Manual, per 2 CCR 2385 (f)(3).

If a Post-Event Inspection is required, an Action Plan shall be submitted to the Division within five (5) days after the event. This deadline may be extended in special circumstances. The Action Plan shall include the scope of the inspection (above water, underwater, electrical, mechanical systems, physical limits, applicable berthing systems, etc.) and submission date of the final report. The Action Plan is subject to Division approval.

2.4.2 Inspection Team

The qualifications of the inspection team shall be the same as those prescribed in Section 2.3.4. Division representatives may participate in any Post-Event Inspection, as observers, to provide guidance when necessary.

2.4.3 Scope

The Post-Event Inspection shall focus on the possible damage caused by the event. General observations of long-term or preexisting deterioration such as significant corrosion-related damage or other deterioration should be made as appropriate, but should not be the focus of the inspection. The Inspection shall always include an above-water assessment of structural, electrical and mechanical components.

The Inspection Team Leader shall determine the need for, and methodology of, an underwater structural assessment, in consultation with the Division. Above water observations, such as shifting or differential settlement, misalignments, significant cracking or spalling, bulging, etc. shall be used to determine whether or not an underwater assessment is required. Similarly, the Inspection Team Leader shall determine, in consultation with the Division, the need for, and methodology of any supplemental inspections (e.g. Special Inspections (2.3.5)).

The following information may be important in determining the need for, and methodology of, the Post-Event Inspection:

- Earthquakes or vessel impact typically cause damage both above and below the water line. Following a major earthquake, the inspection should focus on components likely to attract highest lateral loads (batter or shorter piles, etc.).
- Major floods or tsunamis may cause undermining of the structure, and/or scouring at the mudline.
- Fire damage varies significantly with the type of construction materials but all types may be adversely affected. Special Inspections (sampling and laboratory testing) shall be conducted, as determined by the Inspection Team Leader, in order to determine the nature and extent of damage.
- High wind or wave events often cause damage both above and below the water line. An underwater inspection may be required if damage is visible above the waterline. Structural damage may be potentially increased if a vessel was at the berth during the event. The effects of high wind may be most prevalent on equipment and connections of such equipment to the structure.

The methodology of conducting an underwater Post-Event Inspection should be established with due consideration of the structure type and type of damage anticipated. Whereas slope failures or scour may be readily apparent in waters of adequate visibility, overstressing cracks on piles covered with marine growth will not be readily apparent. Where such hidden damage is suspected, marine growth removal should be performed on a representative sampling of components in accordance with the Level II effort requirements described in Section 2.3.5. However, the sample size and locations shall also consider the cause of the event. Whereas facilities subjected to vessel impact or impact from debris should concentrate the inspection effort on the components in the path of the impact, facilities subjected to strong seismic shaking should concentrate the inspection effort on components likely to attract the most lateral load (batter piles, shorter piles in the rear of the structure, etc.).

2.4.4 Post-Event Ratings

A post-event rating shall be assigned to each berthing system upon completion of the inspection (See Table 2-8). All observations of the above and under water structure, electrical and mechanical components and systems shall be considered in assigning a post-event rating.

Ratings should consider only damage that was likely caused by the event. Pre-existing deterioration such as corrosion damage should not be considered unless the structural integrity is immediately threatened or safety systems or protection of the environment may be compromised.

Assignment of ratings should reflect an overall characterization of the berthing system being rated. The rating shall consider both the severity of the deterioration and the extent to which it is widespread throughout the facility.

Ratings shall be used to describe the existing in place structure and electrical/mechanical systems as compared to the facility when new. The fact that the facility was designed for loads that are lower than the current standards for design should have no influence upon the ratings.

TABLE 2-8 POST-EVENT RATINGS AND ACTIONS		
Rating	**Summary of Damage**	**Remedial Actions**
A	No significant event-induced damage observed.	No further action required. The berthing system may continue operations.
B	Minor to moderate event-induced damage observed but all primary structural elements and electrical/mechanical systems are sound.	Repairs or mitigation may be required to remain operational. The berthing system may continue operations.
C	Moderate to major event-induced damage observed which may have significantly affected the load bearing capacity of primary structural elements or the functionality of key electrical/mechanical systems.	Repairs or mitigation may be necessary to resume or remain operational. The berthing system may be allowed to resume limited operations. (See 2 CCR 2385 (f)).
D	Major event-induced damage has resulted in localized or widespread failure of primary structural components; or the functionality of key electrical/mechanical systems has been significantly affected. Additional failures are possible or likely to occur.	The berthing system may not resume operations until the deficiencies are corrected (See 2 CCR 2385 (f)).

2.4.5 Follow-up Actions

Follow-up actions shall be assigned upon completion of the Post-Event Inspection of each berthing system. Table 2-6 specifies remedial action priorities and actions for electrical and mechanical deficiencies. Table 2-7 specifies various options for structural systems. Multiple follow-up actions may be assigned; however, guidance should be provided as to the order in which the follow-up actions should be carried-out. Follow-up actions shall be subject to Division approval.

2.4.6 Documentation and Reporting

Documentation of the specific attributes of each defect shall not be required during a Post-Event Inspection. However, a narrative description of significant damage shall be used. The description shall be consistent with and shall justify the post-event rating assigned.

A report shall be prepared and submitted to the Division upon completion of the Post-Event Inspection and shall, at a minimum, include:

- Brief description of the facility including the physical limits of the structure, type of construction material(s), and electrical/mechanical systems present.
- Brief description of the event triggering the inspection.
- Scope of the inspection (above water, underwater, electrical or mechanical)
- Date of the inspection
- Names and affiliations of inspection team
- Description of the nature, extent and significance of any observed damage resulting from the event.
- Photographs should be provided to substantiate the descriptions and justify the condition rating
- Assignment of a post-event rating
- Statement regarding whether the facility is fit to resume operations and, if so, under what conditions
- Assignment of follow-up action(s)
- Inspection data, drawings, calculations and other relevant engineering materials
- Signature and stamp of Team Leader(s)

2.4.7 Report of Action Plan

Upon completion of all actions delineated in the Action Plan, a final report shall be submitted to the Division to document the work completed. The report may be brief and shall consist of a description of each action taken, including the reason for the action and the benefit resulting from it. Supporting documentation such as calculations or other relevant data shall be provided in appendices.

2.5 REFERENCE

[2.1] Buslov, V., Heffron, R. and Martirossyan, A., 2001, “Choosing a Rational Sample Size for the Underwater Inspection of Marine Structures,” Proceedings, Ports 2001, ASCE Conference, April 29-May 2, Norfolk, VA.

Placeholder for Figure 2-1

3. STRUCTURAL LOADING CRITERIA

3.1 GENERAL

3.1.1 Purpose

Section 3 establishes the environmental and operating loads acting on the Marine Oil Terminal (MOT) structures and on moored vessel(s). The analysis procedures are presented in Sections 4 – 7.

3.1.2 Applicable Codes, Standards, and Recommended Practice

Structural loading criteria shall conform to the requirements of this section. Additional requirements, as appropriate, are provided in the following codes, regulations, recommended practices and references.

American Petroleum Institute , July, 1993, API RP 2A-LRFD, "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design," Washington, D.C.

American Society of Civil Engineers, Jan. 2000, "Minimum Design Loads for Buildings and Other Structures," ASCE 7-98, Revision of ANSI/ASCE 9-95, Reston, VA.

British Standards Institution, 2000, "British Standard Code of Practice for Maritime Structures - Part 1 General Criteria" BS6349, Part 1, London, England.

Dept. of Defense, 30 June 1994, Military Handbook, "Piers and Wharves," Mil-HDBK-1025/1, Washington, D.C.

Dept. of Defense, 1 July 1999, "Mooring Design," Mil-HDBK-1026/4A, Washington, D.C.

Dept. of the Navy, 1984, "Harbors Design Manual 26.1," NAVFAC DM-26.1, Alexandria, VA.

Federal Emergency Management Agency, FEMA-356, Nov. 2000, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," Washington, D.C.

Ferritto, J., Dickenson, S., Priestley N., Werner, S., Taylor, C., Burke D., Seelig W., and Kelly, S., 1999, "Seismic Criteria for California Marine Oil Terminals," Vol.1 and Vol.2, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, CA.

Pile Buck Production, 1992, "Mooring Systems," Pile Buck Inc., Jupiter, Florida.

Oil Companies International Marine Forum (OCIMF), 1977, "Prediction of Wind and Current Loads on VLCCs," London, England.

Oil Companies International Marine Forum (OCIMF), 1997, "Mooring Equipment Guidelines," 2nd ed., London, England.

3.2 DEAD LOADS

3.2.1 General

The dead loads shall include the weight of entire structure, including permanent attachments such as loading arms, pipe lines, deck crane, fire monitor tower, gangway structure, vapor control equipment and mooring hardware. Loads specified in Sections 3.2.2 and 3.2.3 may be used for MOT structures if actual weights are not available.

3.2.2 Unit Weights

The unit weights in Table 3-1 may be used for both existing and new MOTs:

TABLE 3-1

UNIT WEIGHTS

Material	Unit Weight (pcf)
Steel or cast steel	490
Cast iron	450
Aluminum alloys	175
Timber (untreated)	40-50
Timber (treated)	45-60
Concrete, reinforced (normal weight)	145-160
Concrete, reinforced (lightweight)	90-120
Asphalt paving	150

3.2.3 Equipment and Piping Area Loads

The equipment area loads in Table 3-2 may be used, as a minimum, in lieu of detailed as-built data.

3.3 VERTICAL LIVE LOADS AND BUOYANCY

The following vertical live loading shall be considered, where appropriate:

- Uniform loading
- Truck loading
- Crane loading
- Buoyancy

TABLE 3-2
EQUIPMENT AND PIPING AREA LOADS

Location	Area Loads (psf)
Open areas	20*
Areas containing equipment and piping	35**
Trestle roadway	20*

* Allowance for incidental items such as railings, lighting, miscellaneous equipment, etc.
** 35 psf is for miscellaneous general items such as walkways, pipe supports, lighting, and instrumentation. Major equipment weight shall be established and added into this weight for piping manifold, valves, deck crane, fire monitor tower, gangway structure, and similar major equipment.

In addition, MOT specific, non-permanent equipment shall be identified and used in loading computations.

3.4 EARTHQUAKE LOADS

3.4.1 General

MOTs shall be capable of resisting earthquake motion; considering the distance to active faults, the seismic response of soils at the site and the dynamic response characteristics of the structure. The required level of sophistication in developing the earthquake input motions is dependent on the classification of the MOT (Table 4-1) and local soil conditions.

3.4.2 Design Earthquake Motion Parameters

The earthquake ground motion parameters in terms of spectral and peak ground acceleration, and earthquake magnitude are modified for site amplification and near fault directivity effects. Methodologies to ascertain these parameters are provided in this section. The resulting values of earthquake motion parameters are the Design Peak Ground Acceleration (DPGA) and Design Spectral Acceleration (DSA).

The peak ground motion and spectral acceleration may be evaluated using the U.S. Geological Survey (USGS) or California Geological Survey (CGS, formerly the California Division of Mines and Geology (CDMG)) maps as discussed in Section 3.4.2.2, or a site-specific probabilistic seismic hazard analysis (PSHA) as discussed in Section 3.4.2.3, respectively. For the Ports of Los Angeles, Long Beach and Port Hueneme, a PSHA is provided in Section 3.4.2.4. Unless stated otherwise, the DSA values are for 5 percent damping; values at other levels may be obtained as per Section 3.4.2.9.

The appropriate probability levels associated with DPGA and DSA for different seismic performance levels are provided in Table 4-2. Deterministic earthquake motions, which are used only for comparison to the probabilistic results, are addressed in Section 3.4.2.7.

The evaluation of Design Earthquake Magnitude (DEM), is discussed in Section 3.4.2.8. This parameter is required when acceleration time histories (Section 3.4.2.10) are addressed or if liquefaction potential (Section 6.4) is being evaluated.

3.4.2.1 Site Classes

The following site classes, defined in Section 6.3.1, shall be used in developing values of DSA and DPGA:

S_A, S_B, S_C, S_D, S_E, and S_F.

3.4.2.2 Earthquake Motions from USGS Maps

Earthquake ground motion parameters can be obtained from the maps 29-32 in the National Earthquake Hazard Reduction Program (NEHRP) design map set discussed in Sections 1.6.1 of FEMA-356 [Ref. 3.1], online at (http://geohazards.cr.usgs.gov/eq/html/canvmap.html) or on CD ROM from the USGS. These are available as peak ground acceleration and spectral acceleration values at 5 percent damping for 10 and 2 percent probability of exceedance in 50 years, which correspond to Average Return Periods (ARPs) of 475 and 2,475 years, respectively. The spectral acceleration values are available for 0.2, and 1.0 second spectral periods. In obtaining peak ground acceleration and spectral acceleration values from the USGS web site, the site location can be specified in terms of site longitude and latitude or the zip code when appropriate. The resulting values of peak ground acceleration and spectral acceleration correspond to surface motions for Site Classification approximately corresponding to the boundary of Site Class S_B and S_C.

Once peak ground acceleration and spectral acceleration values are obtained for 10 and 2 percent probability of exceedance in 50 years, the corresponding values for other probability levels may be obtained by interpolation, either analytically or graphically. An example of the analytic method is presented in Section 1.6 of Chapter 1 of FEMA 356 [Ref. 3.1].

In the graphical method, values of spectral acceleration in arithmetic scale may be plotted versus ARP or annual frequency of exceedance values in logarithmic scale for each period. A smooth curve is passed through the data points for interpolation of ground motion values.

Other available maps showing information similar to that of the USGS maps may be used to obtain peak ground acceleration and spectral acceleration values. However, in such cases, justifications for not using USGS values shall be provided.

3.4.2.3 Earthquake Motions from Site-Specific Probabilistic Seismic Hazard Analysis

As an alternative to the method in Section 3.4.2.2, peak ground acceleration and spectral acceleration values may be obtained using site-specific probabilistic seismic hazard analysis (PSHA). In this approach, the seismic sources and their characterization used in the analysis shall be based on the published data from the California Geological Survey (formerly CDMG), which can be obtained online at the following web site: (http://www.consrv.ca.gov/dmg/rghm/psha/Index.htm) [Ref. 3.2]. When seismic source characterization that is significantly different from the CGS (or USGS) data is used, an explanation shall be provided.

When using seismic source characterizations other than the CGS (or USGS) data, appropriate attenuation relationships shall be used to obtain values of peak ground acceleration and spectral acceleration at the ground surface for site conditions corresponding to the boundary of Site Class S_B and S_C, regardless of the actual subsurface conditions at the site. These results shall be compared to those based on the FEMA/USGS maps discussed in Section 3.4.2.2. If the two sets of values are significantly different, a justification for using the characterization chosen shall be provided.

Alternatively, peak ground acceleration and spectral accelerations at the ground surface for the subsurface conditions that actually exist at the site may be directly obtained by using appropriate attenuation relationships in a site-specific PSHA. This approach is not permissible for Site Classes S_E and S_F.

For site-specific PSHA or port PSHA (provided below), peak ground acceleration and spectral acceleration values corresponding to the seismic performance level (See Table 4-2) shall be obtained through probability calculations.

For peak ground acceleration, PSHA may be conducted using the "magnitude weighting" procedure by Idriss [3.3]. The actual magnitude weighting values should follow SCEC [3.4]. This magnitude weighting procedure incorporates the effects of duration corresponding to various magnitude events in the PSHA results. The resulting peak ground acceleration shall be used only for liquefaction assessment (See Section 6.4).

Site-specific hazard information has been developed for the Port of Los Angeles, Port of Long Beach and Port Hueneme. This assessment has included an independent review of onshore and offshore faulting. The work has been performed by Lawrence Livermore National Laboratory [3.5] and the results are provided in Tables 3-3, 3-4 and Figures 3-1 and 3-2. The results are provided only for site classification "S_C" and five percent damping. These spectral values (DSA's) are the minimum acceptable and represent the subsurface only. To obtain appropriate values for piles and/or the mudline, the simplified procedures of Section 3.4.2.4 may be used.

3.4.2.4 Simplified Evaluation of Site Amplification Effects

When the MOT Site Class is different from the S_B - S_C boundary, site amplification effects shall be incorporated in peak ground accelerations and spectral accelerations. This may be accomplished using a simplified method or a site-specific evaluation (Section 3.4.2.5).

For a given Site Class, the following procedure [3.1] presents a simplified method that may be used to incorporate the site amplification effects for peak ground acceleration and spectral acceleration computed for the S_B and S_C boundary.

a) Calculate the spectral acceleration values at 0.20 and 1.0 second period:

$$S_{XS} = F_a S_S \quad (3.1)$$

$$S_{X1} = F_v S_1 \quad (3.2)$$

TABLE 3-3 RESPONSE SPECTRA FOR THE PORTS OF LOS ANGELES AND LONG BEACH (5% CRITICAL DAMPING)		
Site Class "C" (Shear Wave Velocity from 1200-2500 ft/sec)		
Period (Sec)	**Frequency (Hz)**	**Spectral Acceleration (g's)**
0.03	33.33	0.47
0.05	20.00	0.52
0.10	10.0	0.71
0.15	6.67	0.86
0.20	5.0	0.93
0.30	3.33	0.93
0.50	2.00	0.85
1.0	1.0	0.62
2.0	0.50	0.37

TABLE 3-4 RESPONSE SPECTRA FOR PORT HUENEME (5% CRITICAL DAMPING)		
Site Class "C" (Shear Wave Velocity from 1200-2500 ft/sec)		
Period (Sec)	**Frequency (Hz)**	**Spectral Acceleration (g's)**
0.03	33.33	0.41
0.05	20.00	0.46
0.10	10.0	0.63
0.15	6.67	0.75
0.20	5.0	0.80
0.30	3.33	0.78
0.50	2.00	0.69
1.0	1.0	0.49
2.0	0.50	0.28

Where:
F_a and F_v are site coefficients obtained from Tables 3-5 and 3-6, respectively.

S_S = short period (usually at 0.20 seconds) spectral acceleration value (for the boundary of S_B and S_C) obtained using Section 3.4.2.2, or at the period corresponding to the peak in spectral acceleration values when obtained from Section 3.4.2.3

S_1 = spectral acceleration value (for the boundary of S_B and S_C) at 1.0 second period

S_{XS} = spectral acceleration value obtained using the short period S_S and factored by Table 3-5 for the Site Class under consideration.

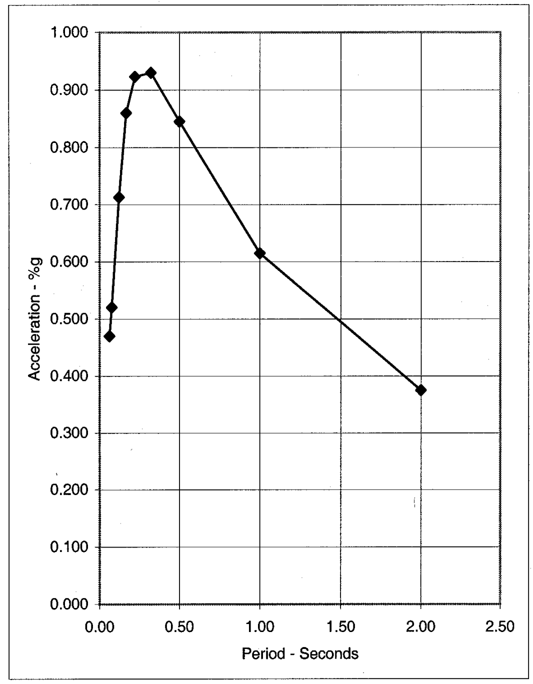

Figure 3-1 Response Spectra for the Ports of Los Angeles and Long Beach (5% Critical Damping)

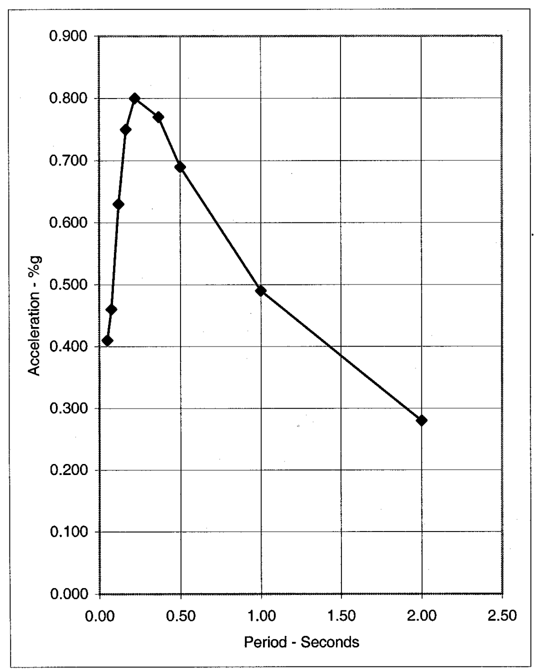

Figure 3-2 Response Spectra for Port Hueneme (5% Critical Damping)

TABLE 3-5
VALUES OF F_a

Site Class	S_S <0.25	0.5	0.75	1.0	> 1.25
S_A	0.8	0.8	0.8	0.8	0.8
S_B	1.0	1.0	1.0	1.0	1.0
S_C	1.2	1.2	1.1	1.0	1.0
S_D	1.6	1.4	1.2	1.1	1.0
S_E	2.5	1.7	1.2	0.9	0.9
S_F	*	*	*	*	*

NOTE: Linear interpolation can be used to estimate values of F_a for intermediate values of S_S.
* Site-specific dynamic site response analysis shall be performed.

TABLE 3-6
VALUES OF F_V

Site Class	S_1 <0.1	0.2	0.3	0.4	>0.5
S_A	0.8	0.8	0.8	0.8	0.8
S_B	1.0	1.0	1.0	1.0	1.0
S_C	1.7	1.6	1.5	1.4	1.3
S_D	2.4	2.0	1.8	1.6	1.5
S_E	3.5	3.2	2.8	2.4	2.4
S_F	*	*	*	*	*

Note: Linear interpolation can be used to estimate values of F_V for intermediate values of S_1.
* Site-specific dynamic site response analysis shall be performed.

S_{X1} = spectral acceleration value obtained using the 1.0 second period S_1 and factored by Table 3-6 for the Site Class under consideration.

b) Set

$$PGA_X = 0.4S_{XS} \tag{3.3}$$

Where:
PGA_X = peak ground acceleration corresponding to the Site Class under consideration.

When the value of PGA_X is less than the peak ground acceleration obtained following Section 3.4.2.2 or Section 3.4.2.3, an explanation of the results shall be provided.

c) PGA_X, S_{XS}, and S_{X1} constitute three spectral acceleration values for the Site Class under consideration corresponding to periods of 0, S_S (usually 0.2 seconds), and 1.0 second, respectively.

d) The final response spectra, without consideration for near-fault directivity effects, values of S_a for the Site Class under consideration may be obtained using the following equations (for 5% critical damping):

for $0<T<0.2T_o$

$$S_a = (S_{XS})(0.4 + 3T/T_o) \tag{3.4}$$

where:
T = Period corresponding to calculated S_a
T_o = Period at which the constant acceleration and constant velocity regions of the design spectrum intersect

for $0.2T_o<T<T_o$

$$S_a = S_{XS} \tag{3.5}$$

for $T>T_o$

$$S_a = S_{X1}/T \tag{3.6}$$

where:

$$T_o = S_{X1}/S_{XS} \tag{3.7}$$

The resulting PGA_X is the DPGA. However, the S_a's (except for the ports of Los Angeles, Long Beach and Port Hueneme) shall be modified for near-fault directivity effects, per Section 3.4.2.6 to obtain the final DSAs.

3.4.2.5 Site-Specific Evaluation of Amplification Effects

As an alternative to the procedure presented in Section 3.4.2.4, a site-specific response analysis may be performed. For S_F, a site specific response analysis is required. The analysis shall be either an equivalent linear or nonlinear analysis. Appropriate acceleration time histories as discussed in Section 3.4.2.10 shall be used.

In general, an equivalent linear analysis using, for example, SHAKE91 [3.6] is acceptable when the strength and stiffness of soils are unlikely to change significantly during the seismic shaking, and the level of shaking is not large. A nonlinear analysis should be used when the strength and/or stiffness of soils could significantly change during the seismic shaking or significant non-linearity of soils is expected because of high seismic shaking levels.

The choice of the method used in site response analysis shall be justified considering the expected stress-strain behavior of soils under the shaking level considered in the analysis.

Site-specific site response analysis may be performed using one-dimensional analysis. However, to the extent that MOTs often involve slopes or earth retaining structures, the one-dimensional analysis should be used judiciously. When one-dimensional analysis cannot be justified or is not adequate, two-dimensional equivalent linear or nonlinear response analysis shall be performed. Site-specific response analysis results shall be compared to those based on the simplified method of Section 3.4.2.4 for reasonableness.

For the port areas of Los Angeles, Long Beach and Port Hueneme, the resulting response spectra shall not fall below values obtained using the simplified method of Section 3.4.2.4.

The peak ground accelerations obtained from this site-specific evaluation are DPGAs and the spectral accelerations are DSAs as long as the near-fault directivity effects addressed in Section 3.4.2.6 are appropriately incorporated into the time histories (Section 3.4.2.10).

3.4.2.6 Directivity Effects

When the site is 15 km (9.3 miles) or closer to a seismic source that can significantly affect the site, near-fault directivity effects shall be reflected in the spectral acceleration values. However, Tables 3-3 and 3-4 for the port areas of Los Angeles, Long Beach and Port Hueneme already have these effects included.

Directivity effects can be incorporated in two possible methods. In the first method, the effects may be reflected in the spectral acceleration values in a deterministic manner by using, for example, the equation on pg. 213 (and Tables 6 and 7) of Somerville, et al. [3.7]. The critical seismic sources and their characterization developed as part of the deterministic ground motion parameters (Section 3.4.2.7) should be used to evaluate the directivity effects. The resulting adjustments in spectral acceleration values may be applied in the probabilistic spectral acceleration values developed per Section 3.4.2.4 or Section 3.4.2.5. Such adjustment can be independent of the probability levels of spectral accelerations.

Secondly, the directivity effects may be incorporated in the results of site-specific PSHA per Section 3.4.2.3. In this case, the directivity effects will also depend on the probability level of spectral accelerations.

If spectral accelerations are obtained in this manner, the effects of site amplification using either Section 3.4.2.4, 3.4.2.5 or an equivalent method (if justified) shall be incorporated.

3.4.2.7 Deterministic Earthquake Motions

Deterministic ground motions from "scenario" earthquakes may be used only for comparison purposes. Deterministic peak ground accelerations and spectral accelerations may be obtained using the "Critical Seismic Source" with its closest appropriate distance to the MOT and the maximum earthquake magnitude. "Critical Seismic Source" is that which results in the largest computed median peak ground acceleration and spectral acceleration values when appropriate attenuation relationships are used. The values obtained from multiple attenuation relationships should be used to calculate the median peak ground acceleration and spectral acceleration values.

Seismic sources shall be identified and characterized based on the CDMG (or USGS) information. When more than one seismic source results in the largest peak ground accelerations and spectral accelerations depending on the period, the appropriate critical sources for particular structures and conditions should be selected.

When the MOT is 15 km (9.3 miles) or closer to a seismic source that can significantly affect the site, near-fault directivity effects should be reflected in the deterministic spectral acceleration values following the procedures in Section 3.4.2.6.

Alternatively, the median deterministic values of peak ground accelerations and spectral accelerations may be obtained, for example, from the USGS maps [3.1] showing the peak ground acceleration and spectral accelerations corresponding to the Maximum Considered Earthquake (MCE). In this case, the actual values of median deterministic peak ground acceleration and spectral acceleration should be 2/3

(See Section 1.6, Ref [3.1]) of the values shown on the USGS maps.

3.4.2.8 Design Earthquake Magnitude

The Design Earthquake Magnitude used in developing site-specific acceleration time histories (Section 3.4.2.10) or liquefaction assessment (Section 6.4) is defined by a Magnitude and a Distance from the site, and shall be obtained using either of the following two methods.

1) As part of the deterministic ground motion parameters, the Design Earthquake may be selected as the largest earthquake magnitude associated with the Critical Seismic Source. The distance shall be taken as the closest distance from the source to the site. The resulting Design Earthquake shall be associated with all DPGA values for the site, irrespective of probability levels.

2) The Design Earthquake (DEQ) may be obtained for each DPGA or DSA value and associated probability level by determining the corresponding dominant distance and magnitude. These are the values of the distance and magnitude that contribute the most to the seismic mean seismic hazards estimates for the probability of interest. They are usually determined by locating the summits of the 3-D surface of contribution of each small interval of magnitude and distance to the total mean hazards estimate. If this 3-D surface shows several modes with approximate weight of more than 20% of the total, several DEQs may be considered, and the DEQ leading to the most conservative design parameters shall be used.

3.4.2.9 Design Spectral Acceleration for Various Damping Values

Design Spectral Acceleration (DSA) values at damping other than 5% shall be obtained by using a current procedure [3.1], and denoted as DSA_d. Such a procedure shall incorporate the near-fault directivity effects when the MOT is 15 km (9.3 miles) or closer to a significant seismic source. One procedure that does not include near-fault directivity effects, but may be adequate for many conditions is as follows:

For $0 < T < 0.2\,T_o$

$$DSA_d = S_{XS}\ [\ (5/B_S - 2)\ T/\ T_o + 0.4] \tag{3-8}$$

For $0.2T_o < T < T_o$

$$DSA_d = DSA/\ B_S \tag{3.9}$$

For $T > T_o$

$$DSA_d = S_1\ /(B_1\ T) \tag{3.10}$$

where:

T = period

T_o = S_{X1}/S_{XS}

B_S = Coefficient used to adjust the short period spectral response, for the effect of viscous damping.

B_1 = Coefficient used to adjust one-second period spectral response, for the effect of viscous damping

Values of B_S and B_1 are obtained from Table 3-7

TABLE 3-7

VALUES OF B_S AND B_1

Damping (%)	B_S	B_1
<2	0.8	0.8
5	1.0	1.0
10	1.3	1.2
20	1.8	1.5
30	2.3	1.7
40	2.7	1.9
>50	3.0	2.0

Note: Linear interpolation should be used for damping values not specifically listed.

3.4.2.10 Development of Acceleration Time Histories

When acceleration time histories are utilized, target spectral acceleration values shall be initially selected corresponding to the DSA values at appropriate probability levels. For each set of target spectral acceleration values corresponding to one probability level, at least three sets of horizontal time histories (one or two horizontal acceleration time histories per set) shall be developed.

Appropriate initial time histories shall have magnitude, distance, and the type of fault that are reasonably similar to those associated with the conditions contributing most to the probabilistic DSA values. Preferred initial time histories should have their earthquake magnitude and distance to the seismic source similar to the mode-magnitude and mode-distance from the PSHA or from appropriate maps. When an adequate number of appropriate recorded time histories are not available, acceleration time histories from simulation may be used as supplements.

Scaling or adjustments, either in the frequency domain or in the time domain (preferably), prior to generating acceleration time histories should be kept to a minimum. When the target spectral accelerations reflect near-fault directivity effects (Section 3.4.2.6), the initial time histories should exhibit directivity effects.

When three sets of time histories are used in the analysis, the envelope of the spectral acceleration values from each time history shall be equal to or higher than the target spectral accelerations. If the envelope values fall below the target values, adjustments shall be made to insure that the spectral acceleration envelope is higher than target spectral accelerations. If the envelope is not higher, then a justification shall be provided.

When seven or more sets of time histories are used, the average of the spectral acceleration values from the set of time histories shall be equal or higher than the target spectral acceleration values. If the average values fall below the target values, adjustments shall be made to insure that average values are higher than the target spectral accelerations. If this is not the case, then an explanation for the use of these particular spectral acceleration values shall be provided.

When three sets of time histories are used in the analysis, the maximum value of each response parameter shall be used in the design/evaluation/rehabilitation efforts. When seven or more sets of time histories are used in the analysis, the average value of each response parameter may be used.

3.5 MOORING LOADS

3.5.1 General

Forces acting on a moored vessel may include: wind, current, waves, hydrodynamic forces induced by passing vessels, tidal variations, seiche and tsunamis. Forces from wind and current acting directly on the structure (not through the vessel in the form of mooring and/or breasting loads) shall be determined from Section 3.7. The vessel's moorings shall satisfy 2 CCR 2340 (c) (1).

3.5.2 Wind Loads

Loads induced on vessels by wind, while moored at an on-shore MOT shall be calculated using procedures described in this section. Wind loads shall be calculated for each of the load cases identified in Section 5.3.

3.5.2.1 Design Wind Speed

The design wind speed is the maximum wind speed of 30 second duration used in the mooring analysis (See Section 5). The mooring risk classification, based on the design wind speed is shown in Table 5-1.

Operating Condition:

An operating condition is the safe wind envelope derived from the mooring analysis (See Section 5). This is the design wind speed below which a vessel may conduct transfer operations. When this maximum operating wind condition is exceeded, at an existing MOT, a vessel is required to cease transfer operations.

Survival Condition:

The survival condition at a new MOT is defined as the state wherein a vessel can remain safely moored at the berth during severe winds. The survival condition threshold is the maximum wind velocity, for a 30 second gust and a 25 year return period, obtained from historical data.

For an existing MOT, a reduced survival condition is acceptable, above which the vessel must leave the berth, within 30 minutes or less (see 2 CCR 2340 (c) (28)).

The 30-second duration wind speed shall be determined from the annual maximum wind data. Average annual summaries cannot be used. Maximum wind speed data for eight directions (45-degree increments) shall be obtained. If other duration wind data is available, it shall be adjusted to a 30-second duration, in accordance with Section 3.5.2.2. The 25-year return period shall be used to establish the design wind speed for each direction. Once these wind speeds are established, the highest wind speed shall be used to determine the mooring risk classification, as shown in Table 5-1. For barges and non-tank vessels, the methods outlined in ASCE 7 (see 3.1.2) may be used.

3.5.2.2 Wind Speed Corrections

Wind speed measured at an elevation of 33 feet (10 meters) above the water surface, with duration of 30 seconds shall be used to determine the design wind speed. If these conditions are not met, the following corrections shall be applied. The correction for elevation is obtained from the equation:

$$V_w = V_h \left(\frac{33}{h}\right)^{1/7} \tag{3.11}$$

where:

V_w = wind speed at elevation 33 ft. (10 m.)
V_h = wind speed at elevation h
h = elevation above water surface of wind data[feet]

The available wind duration shall be adjusted to a 30-second value, using the following formula:

$$V_{t=30\sec} = \frac{v_t}{c_t} \tag{3.12}$$

where:

$V_{t=30sec}$ = wind speed for a 30 second duration
v_t = wind speed over a given duration
c_t = conversion factor from Figure 3-3

If wind data is available over land only, the following equation shall be used to convert the wind speed from over-land to over-water conditions ("Mooring Systems", Pile Buck, see 3.1.2):

$$V_w = 1.10\ V_L \tag{3.13}$$

where:

V_w = over water wind speed
V_L = over land wind speed

3.5.2.3 Static Wind Loads on Vessels

The "Prediction of Wind and Current Loads on VLCC's"(OCIMF) or the "British Standard Code of Practice for Maritime Structures" (see 3.1.2) shall be used to determine the wind loads for all tank vessels. For barges and other vessels with configurations different from tankers, the wind loads may be calculated based on the guidelines in ASCE 7.

Alternatively, wind loads for any type of vessel may be calculated using the guidelines in Ferritto et al, 1999, "Seismic Criteria for California Marine Terminals, Volume 2, Appendix B: Mooring Design and Inspection Criteria" (see 3.1.2).

3.5.3 Current Loads

Environmental loads induced by currents at an onshore MOT shall be calculated as specified in this section.

3.5.3.1 Design Current Velocity

Maximum ebb and flood currents, annual river runoffs and controlled releases shall be considered when establishing the design current velocities for both existing and new MOTs.

Local current velocities may be obtained from NOAA [3.8] or other sources, but must be supplemented by site-specific data, if the MOT is classified as high risk based on the maximum current velocities of Table 5-1. Site specific data shall be obtained by real time measurements over a one-year period. If this information is not available, a safety factor of 1.25 shall be applied to the best obtainable data until real time measurements are obtained.

If the facility is not in operation during annual river runoffs and controlled releases, the current loads may be adjusted. Operational dates need to be clearly stated in the definition of the terminal operating limits (see Section 2.3.6).

3.5.3.2 Current Velocity Adjustment Factors

An average current velocity (V_c) shall be used to compute forces and moments. If the vertical current velocity profile is known, the definition of the average current velocity can be obtained from the following equation:

$$V_c^2 = 1/T \int_O^T (V_c)^2\, ds \tag{3.14}$$

where:

V_c = average current velocity [knots]

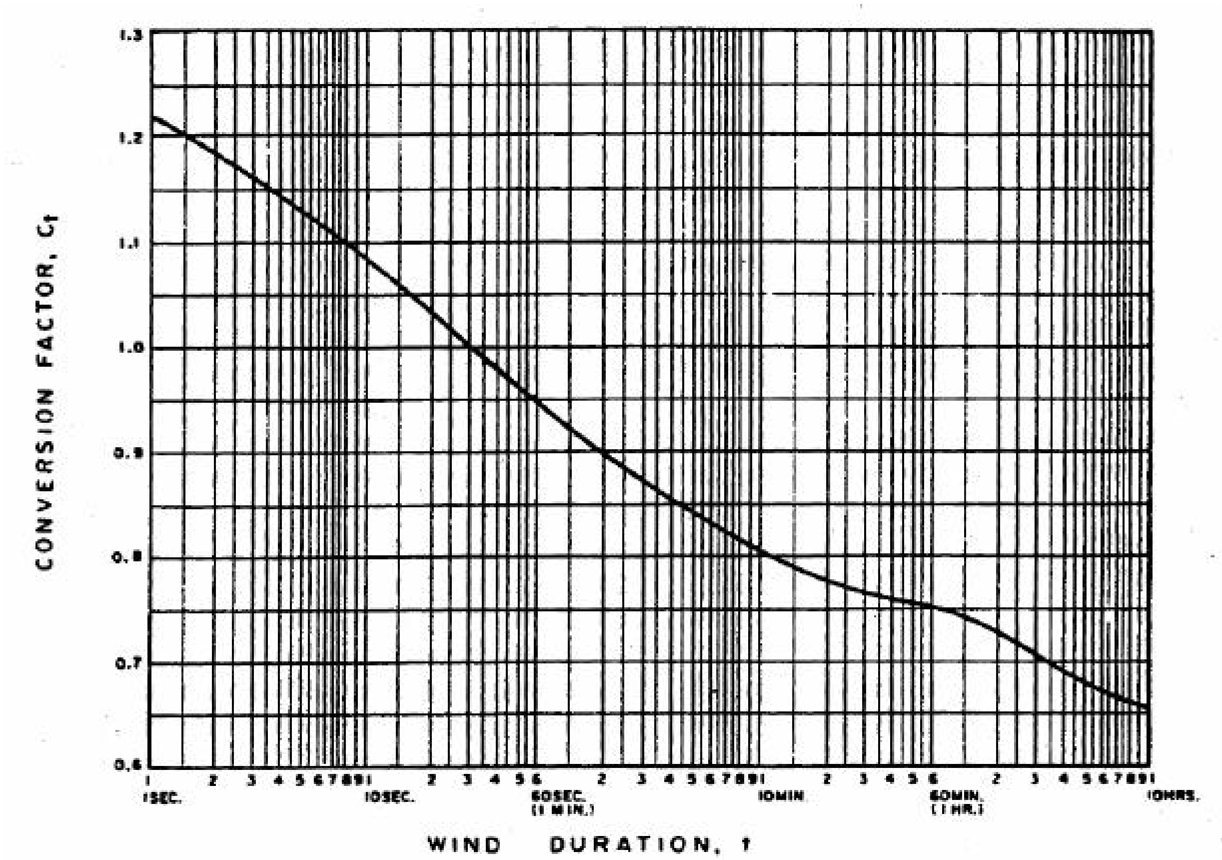

Figure 3-3 Windspeed Conversion Factor ("Mooring Systems", Pile Buck)

T = draft of vessel [meters]
v_c = current velocity as a function of depth [knots]
s = water depth measured from the surface [meters]

If the velocity profile is not known, the velocity at a known water depth should be adjusted by the factors provided in Figure 3-4 to obtain the equivalent average velocity over the draft of the vessel.

3.5.3.3 Static Current Loads on Vessels

The OCIMF or the British Standard procedures shall be used to determine current loads for all moored tank vessels. Current loads for any type of vessel may be calculated using the guidelines in the Dept. of Defense, Mil-HDBK-1026/4A (see 3.1.2).

3.5.4 Wave Loads

When required in accordance with Section 5.2, the transverse wave induced vessel reactions shall be calculated using a simplified dynamic mooring analysis described below.

The horizontal water particle accelerations shall be calculated for the various wave conditions, taken at the mid-depth of the loaded vessel draft. The water particle accelerations shall then be used to calculate the wave excitation forces to determine the static displacement of the vessel. The Froude-Krylov method [3.9] may be used to calculate the wave excitation forces, by conservatively approximating the vessel as a rectangular box with dimensions similar to the actual dimensions of the vessel. The computed excitation force assumes a 90 deg incidence angle with the longitudinal axis of the vessel, which will result in forces that are significantly greater than the forces that will actually act upon the vessel from quartering seas. A load reduction factor may be used to account for the design wave incidence angle from the longitudinal axis of the ship. The overall excursion of the vessel shall be determined for each of the wave conditions by calculating the dynamic response of the linear spring mass system. The corresponding fender reactions shall be calculated from the fender unit load-excursion curves.

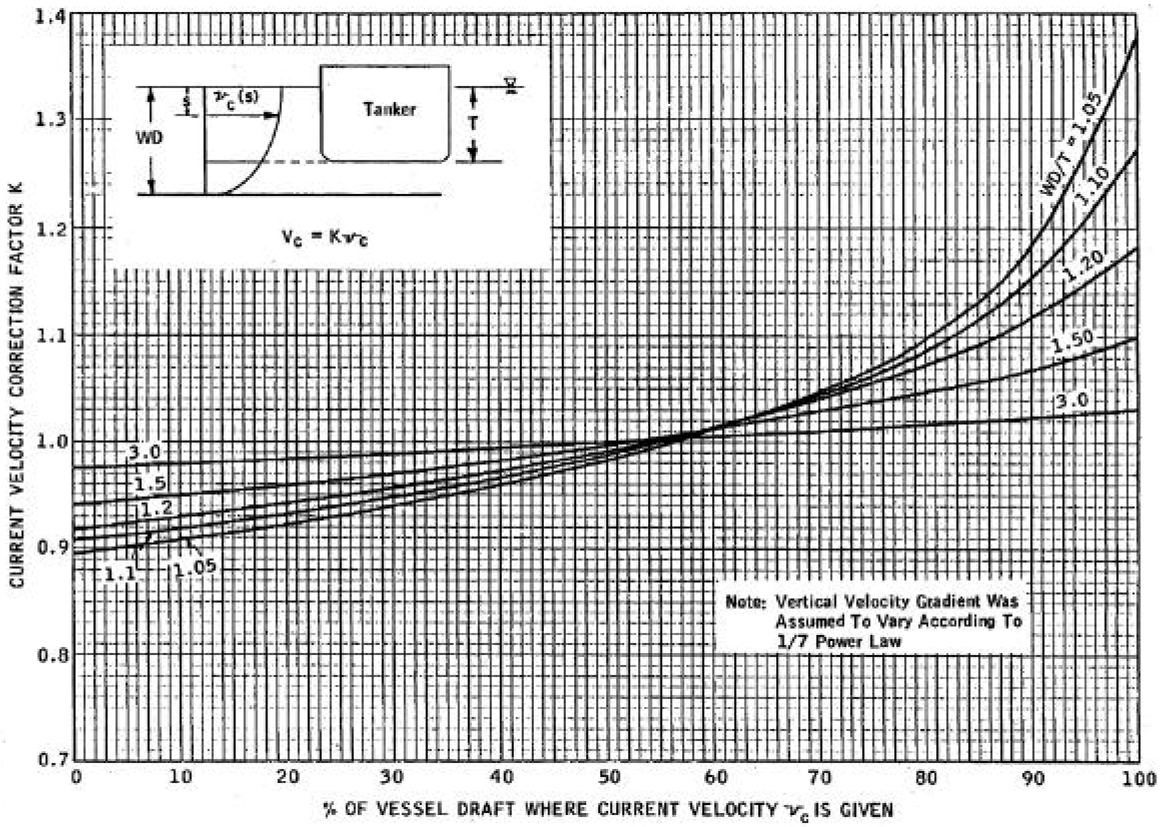

Figure 3-4 Current Velocity Correction Factor ("Prediction of Wind and Current Loads on VLCCs," p. 41, OCIMF

3.5.5 Passing Vessels

When required in Section 5.2, the sway and surge forces, as well as yaw moment, on a moored vessel, due to passing vessels, shall be established considering the following:

- Ratio of length of moored vessel to length of passing vessel
- Distance from moored vessel to passing vessel
- Ratio of mid-ship section areas of the moored and passing vessels
- Underkeel clearances of the moored and passing vessels
- Draft and trim of the moored vessel and draft of the passing vessel
- Mooring line tensions

When current is present, the passing vessel speed must consider the ebb and flood current. Thus, moving against the current will increase the force.

Normal operating wind and current conditions can be assumed when calculating forces due to a passing vessel. Three methods to determine forces on a moored vessel, subjected to passing vessel loads are as follows:

- A simplified theoretical analysis by Wang [3.10] may be used to evaluate the surge and sway forces and yaw moment. Wang developed graphs of non-dimensional surge and sway forces and yaw moment as a function of the separation between the moored and passing vessels and the effect of water depth.
- A second method, developed by Flory [3.11] can be used to calculate the surge and sway forces and yaw moment.
- A third simplified approach has been formulated by Seelig [3.12].

3.5.6 Seiche

The penetration of long period / low amplitude waves into a harbor can result in resonant standing wave systems, when the wave forcing frequency coincides with a natural frequency of the harbor. The resonant standing waves can result in large surge motions if this frequency is close to the natural frequency of the mooring system. Section 5.3.3 prescribes the procedure for the evaluation of these effects.

3.5.7 Tsunamis

A tsunami may be generated by an earthquake or a subsea or coastal landslide, which may induce large wave heights and excessive currents. The large wave or surge and the excessive currents are potentially damaging to an MOT, especially if there is a tank vessel moored alongside. Table 3.8 provides estimated tsunami run-up values for specific areas of California.

TABLE 3-8
TSUNAMI RUN-UP VALUES [FT] IN CALIFORNIA [3.13], [3.14]

Location	Return Period 100 years	Return Period 500 years
W. Carquinez Strait	3.3	4.0
Richmond Harbor Channel	7.6	13.5
Richmond Inner Harbor	5.9	10.6
Oakland Inner Harbor	4.7-5.5	7.5-9.5
Oakland Middle Harbor	5.9	10.5
Oakland Outer Harbor	7.9-9.1	15.1-17.6
Hunters Point	3.9-5.3	5.0-8.7
San Francisco – S. of Bay Bridge	4.5-5.0	7.5-8.4
Ports of Los Angeles and Long Beach	8.0	15.0
Port Hueneme	11.0	21.0

Tsunamis can be generated either by a distant or near source. A tsunami generated by a distant source (far field event) may allow operators to have an adequate warning for mitigating the risk by departing the MOT and going into deep water. For near-field events, with sources less than 500 miles away, the vessel may not have adequate time to depart (see Section 5.3.4).

Loads from tsunami-induced waves can be calculated for various structural configurations [3.15]. Tsunami wave heights in shallow water and particle kinematics can also be obtained. Other structural considerations include uplift and debris impact.

3.6 BERTHING LOADS

3.6.1 General

Berthing loads are quantified in terms of transfer of kinetic energy of the vessel into potential energy dissipated by the fender(s). The terms and equations below are based on those in Mil-HDBK-1025/1, "Piers and Wharves"(see 3.1.2).

Kinetic energy shall be calculated from the following equation:

$$E_{vessel} = \frac{1}{2} \cdot \frac{W}{g} \cdot V_n^{\,2} \tag{3.15}$$

where:
E_{vessel} = Berthing energy of vessel [ft-lbs]
W = Displacement of the vessel in pounds [long tons x 2240]
g = Acceleration due to gravity [32.2 ft/sec^2]
V_n = Berthing velocity normal to the berth [ft/sec]

The following correction factors shall be used to modify the actual energy to be absorbed by the fender system:

$$E_{fender} = C_b \cdot C_m \cdot E_{vessel} \tag{3.16}$$

where:
E_{fender} = Energy to be absorbed by the fender system
C_b = Berthing Coefficient
C_m = Effective mass or virtual mass coefficient (see 3.6.6)

The berthing coefficient, C_b, consists of four terms:

$$C_b = C_e \cdot C_g \cdot C_d \cdot C_c \tag{3.17}$$

where:
C_e = Eccentricity Coefficient
C_c = Configuration Coefficient
C_g = Geometric Coefficient
C_d = Deformation Coefficient

These coefficients are defined in sections 3.6.2 through 3.6.5.

The approximate displacement of the vessel (when only partially loaded) at impact, DT, can be determined from an extension of an equation from Gaythwaite [3.16]:

$$DT = 1.25 DWT \left(d_{actual} / d_{\max} \right) \tag{3.18}$$

where:
DWT = Dead Weight Tonnage (in long tons)

d_{actual} = Actual arrival draft of the vessel
d_{max} = Maximum loaded vessel draft

The berthing load shall be based on the fender reaction due to the kinetic berthing energy. The structural capacity shall be established based on allowable concrete, steel or timber properties in the structural components, as defined in Section 7.

3.6.2 Eccentricity Coefficient (C_e)

During the berthing maneuver, when the vessel is not parallel to the berthing line (usually the wharf face), not all the kinetic energy of the vessel will be transmitted to the fenders. Due to the reaction from the fender(s), the vessel will start to rotate around the contact point, thus dissipating part of its energy. Treating the vessel as a rigid rod of negligible width in the analysis of the energy impact on the fenders leads to the equation:

$$C_e = \frac{k^2}{a^2 + k^2} \tag{3.19}$$

where:
k = Longitudinal radius of gyration of the vessel [ft]
a = Distance between the vessel's center of gravity and the point of contact on the vessel's side, projected onto the vessel's longitudinal axis [ft]

3.6.3 Geometric Coefficient (C_g)

The geometric coefficient, C_g, depends upon the geometric configuration of the ship at the point of impact. It varies from 0.85 for an increasing convex curvature to 1.25 for concave curvature. Generally, 0.95 is recommended for the impact point at or beyond the quarter points of the ship, and 1.0 for broadside berthing in which contact is made along the straight side (Mil-HDBK-1025/1, Piers and Wharves).

3.6.4 Deformation Coefficient (C_d)

This accounts for the energy reduction effects due to local deformation of the ships hull and deflection of the whole ship along its longitudinal axis. The energy absorbed by the ship depends on the relative stiffness of the ship and the obstruction. The deformation coefficient varies from 0.9 for a nonresilient fender to nearly 1.0 for a flexible fender. For larger ships on energy-absorbing fender systems, little or no deformation of the ship takes place; therefore, a coefficient of 1.0 is recommended.

3.6.5 Configuration Coefficient (C_c)

This factor accounts for the difference between an open pier or wharf and a solid pier or wharf. In the first case, the movements of the water surrounding the berthing vessel are not (or hardly) affected by the berth. In the second case, the water between the berthing vessel and the structure, introduces a cushion effect that represents an extra force on the vessel away from the berth and reduces the energy to be absorbed by the fender system.

For open berth and corners of solid piers, $C_c = 1.0$

For solid piers with parallel approach, $C_c = 0.8$

For berths with different conditions, C_c may be interpolated between these values (Mil-HDBK-1025/1, Piers and Wharves).

3.6.6 Effective Mass or Virtual Mass Coefficient (C_m)

In determining the kinetic energy of a berthing vessel, the effective or the virtual mass is the sum of vessel mass and hydrodynamic mass. The hydrodynamic mass does not necessarily vary with the mass of the vessel, but is closely related to the projected area of the vessel at right angles to the direction of motion. Other factors, such as the form of vessel, water depth, berthing velocity, and acceleration or deceleration of the vessel, will have some effect on the hydrodynamic mass. Taking into account both model and prototype experiments, the effective or virtual mass coefficient can be estimated as:

$$C_m = 1 + 2 \cdot \frac{d_{actual}}{B} \tag{3.20}$$

where:
d_{actual} = Actual arrival draft of the vessel
B = Beam of vessel

The value of C_m for use in design should be a minimum of 1.5 and need not exceed 2.0 (Mil-HDBK-1025/1, Piers and Wharves).

3.6.7 Berthing Velocity and Angle

The berthing velocity, V_n, is influenced by a large number of factors such as, environmental conditions of the site (wind, current, and wave), method of berthing (with or without tug boat assistance), condition of the vessel during berthing (ballast or fully laden), and human factors (experience of the tug boat captain.). The berthing velocity, normal to berth, shall be in accordance with Table 3-9, for existing berths and the site condition in accordance with Table 3-10. For new berths, the berthing velocity, V_n, is per PIANC guidelines, Table 4.2.1 [3.17].

For existing MOTs, if it can be demonstrated that lower velocities can be obtained and verified by velocity monitoring equipment, then such a velocity may be used, subject to Division approval.

In order to obtain the normal berthing velocity V_n, approach angles, defined as the angle formed by the fender line and the longitudinal axis of the vessel must be considered. The berthing angles, used to compute the normal velocity, for various vessel sizes are shown in Table 3-11.

3.7 WIND AND CURRENT LOADS ON STRUCTURES

3.7.1 General

This section provides methods to determine the loads acting on the structure directly, as opposed to forces acting on the structure from a moored vessel.

3.7.2 Wind Loads

ASCE 7 shall be used to establish minimum wind loads on the structure. Supplemental calculations for wind loads may be obtained from Reference [3.18].

3.7.3 Current Loads

The current forces acting on the structure may be established using the current velocities, per Section 3.5.

3.8 LOAD COMBINATIONS

Each component of the structure shall be analyzed for all applicable load combinations given in Tables 3-12 and 3-13. These tables provide the load combinations for Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) approaches. One of these two tables is required, depending on component type, see MIL-HDBK-1025/1 (Section 3.1.2).

For example, concrete and steel piles are evaluated using LRFD, whereas mooring fittings, bolts, etc, shall be evaluated using ASD. Mooring lines are controlled by safety factors against the minimum breaking strength of the line.

The various load types used in the load combination equations are discussed below:

Dead Load (D)

Upper and lower bound values of dead load are applied for the normal condition to check the maximum moment and shear with minimum axial load.

Live Load (L)

The live load on MOTs is typically very small and is therefore neglected for all load combinations having earthquake loads.

TABLE 3-9

BERTHING VELOCITY V_n (NORMAL TO BERTH)

Vessel Size (dwt)	Tug Boat Assistance	Site Conditions		
		Very Unfavorable	Moderate	Favorable
<10,000[1]	No	1.31 ft/sec	0.98 ft/sec	0.53 ft/sec
10,000 – 50,000	Yes	0.78 ft/sec	0.66 ft/sec	0.33 ft/sec
50,000 – 100,000	Yes	0.53 ft/sec	0.39 ft/sec	0.33 ft/sec
>100,000	Yes	0.39 ft/sec	0.33 ft/sec	0.33 ft/sec

1. If tug boat is used for vessel size smaller than 10,000 DWT the berthing velocity may be reduced by 20%

TABLE 3-10
SITE CONDITIONS

Site Conditions	Description	Wind Speed[1]	Significant Wave Height	Current Speed[2]
Very Unfavorable	Strong Wind Strong Currents High Waves	>38 knots	>6.5 ft	>2 knots
Moderate	Strong Wind Moderate Current Moderate Waves	>38 knots	<6.5	<2 knots
Favorable	Moderate Wind Moderate Current Moderate Waves	<38 knots	<6.5	<2 knots

1. A 30-second duration measured at a height of 33 ft.
2. Taken at 0.5 x water depth

TABLE 3-11
BERTHING ANGLE

Vessel Size (DWT)	Angle [degrees]
Barge	15
<10,000	10
10,00-50,000	8
> 50,000	6

Buoyancy Load (B)

Buoyancy forces shall be considered for any submerged or immersed substructures (including pipelines, sumps and structural components).

Wind and Currents on the Structure (W, C)

Wind and currents on the vessel are included in the mooring and breasting loads. The wind and current loads acting on the structure are therefore additional loads that can act simultaneously with the mooring, breasting and/or berthing loads.

Earth Pressure at End Wall (H)

The soil pressure on end walls, typically concrete cut-off walls or steel sheet pile walls on wharf type structures, shall considered.

Mooring Line/Breasting Loads (M)

Mooring line and breasting loads can occur simultaneously or individually, depending on the combination of wind and current. Multiple load cases for operating and survival conditions may be required.

TABLE 3-12
LRFD LOAD FACTORS FOR LOAD COMBINATIONS

Load Type	Normal Condition	Mooring & Breasting Condition	Berthing Condition	Earthquake Condition
Dead Load (D)	1.4[a]	1.2	1.2	$1 \pm k$[c]
Live Load (L)	1.7[b]	1.7[b]		
Buoyancy (B)	1.3	1.3	1.3	
Wind on Structure (W)	1.3	1.3	1.0	
Current on Structure (C)	1.3	1.3	1.0	
Earth Pressure at End Wall (H)	1.6	1.6	1.6	1.0
Mooring/Breasting Load (M)		1.3		
Berthing Load (B_e)			1.7	
Earthquake Load (E)				1.0

a. Reduce load factor for dead load (D) to 0.9 to check components for minimum axial load and maximum moment.
b. The load factor for live load (L) may be reduced to 1.3 for the maximum outrigger float load from a truck crane.
c. k = 0.50 (PGA)

TABLE 3-13 ASD LOAD FACTORS FOR LOAD COMBINATIONS				
Load Type	**Normal Condition**	**Mooring & Breasting Condition**	**Berthing Condition**	**Earthquake Condition**
Dead Load (D)	1.0	1.0	1.0	$1 \pm k^{c}$
Live Load (L)	1.0	1.0		
Buoyancy (B)	1.0	1.0	1.0	
Wind on Structure (W)	1.0	1.0	1.0	
Current on Structure (C)	1.0	1.0	1.0	
Earth Pressure at End Wall (H)	1.0	1.0	1.0	1.0
Mooring/Breasting Load (M)		1.0		
Berthing Load (B_e)			1.0	
Earthquake Load (E)				1.0

c. k= 0.5 (PGA)

In addition, loads caused by passing vessels shall be considered for the operational condition. Refer to Section 5 for the determination of mooring line and breasting loads.

Berthing Load (B_e)

Berthing is a frequent occurrence, and shall be considered as a normal, operating load. No allowable increase in capacity shall be applied for ASD, and a load factor of 1.7 shall be applied for the LRFD approach.

Earthquake Loads (E)

A load factor of 1.0 shall be assigned to the earthquake loads. The performance based seismic analysis methodology described in Sections 4 and 7 require that the actual force demand be limited to defined concrete, steel and timber strains. For the deck and pile evaluation, two cases of dead load (upper and lower bound) shall be considered in combination with the seismic load. The upper and lower bound dead load values are expressed in relation to the peak ground acceleration (PGA) such that the load factor becomes (1±k) D, where k = 0.5 (PGA).

3.9 SAFETY FACTORS FOR MOORING LINES

Safety factors for different material types of mooring lines are given in Table 3-14. The safety factors should be applied to the minimum number of lines specified by the mooring analysis, using the highest loads calculated for the environmental conditions. The Minimum Breaking Load (MBL) of new ropes is obtained from the certificate issued by the manufacturer. If nylon tails are used in combination with steel wire ropes, the safety factor shall be based on the weaker of the two ropes.

TABLE 3-14 SAFETY FACTORS FOR ROPES*	
Rope Type	**Safety Factor on Dry MBL**
Steel Wire Rope	1.82
Nylon	2.2
Other Synthetic	2.0
Polyester Tail	2.3
Nylon Tail	2.5

*From Mooring Equipment Guidelines, OCIMF

3.10 MOORING HARDWARE

For new MOTs, a minimum of three hooks are required for each breasting line location for tankers larger than 50,000 DWT. At least two hooks at each location shall be provided for breasting lines for tankers less than 50,000 DWT.

All hooks (new and existing MOTs) shall withstand the minimum breaking load (MBL) of the strongest line with a Safety Factor of 1.2 or greater. Only one mooring line shall be placed on each quick release hook.

3.10.1 Fittings

Marine hardware consists of mooring fittings and base bolts. Mooring fittings consist of cleats, bitts, bollards, and quick release hook assemblies.

The certificate issued by the manufacturer normally defines the allowable working capacity of the marine fitting. If the allowable working loads are not available, the typical values listed in Table 3-15 may be used, for typical sizes, bolt patterns and layout. The allowable working loads are defined for mooring line angles up to 60 degrees from the horizontal. The combination of vertical and horizontal loads must be considered.

TABLE 3-15

ALLOWABLE WORKING LOADS

Type of Fittings	No. of Bolts	Bolt Size [in]	Working Load [kips]
30 in. Cleat	4	1-1/8	20
42 in. Cleat	6	1-1/8	40
Low Bitt	10	1-5/8	60 per column
High Bitt	10	1-3/4	75 per column
44-1/2 in. Ht. Bollard	4	1-3/4	70
44-1/2 in. Ht. Bollard	8	2-1/4	200
48 in. Ht. Bollard	12	2-3/4	450

Note: This table is modified from MIL-HDBK-1026/4A, Table 48

3.10.2 Base Bolts

Base bolts are subjected to both shear and uplift. Forces on bolts shall be determined using the following factors:

- Height of load application on bitts or bollards
- Actual vertical angles of mooring lines for the highest and lowest tide and vessel draft conditions, for all sizes of vessels at each particular berth
- Actual horizontal angles from the mooring line configurations, for all vessel sizes and positions at each particular berth
- Simultaneous loads from more than one vessel

For existing MOTs, the deteriorated condition of the base bolts and supporting members shall be considered in determining the capacity of the fitting.

3.11 SYMBOLS

a	=	Distance between the vessel's center of gravity and the point of contact on the vessel's side, projected onto the vessel's longitudinal axis [ft]
B	=	Beam of vessel
B_1, B_s	=	Damping adjustment factors according to Table 3-7.
C_b	=	Berthing Coefficient
C_c	=	Configuration Coefficient
C_g	=	Geometric Coefficient
C_d	=	Deformation Coefficient
C_e	=	Eccentricity Coefficient
C_m	=	Effective mass or virtual mass coefficient
C_t	=	Windspeed conversion factor
DEM	=	Design Earthquake Magnitude
$DPGA$	=	Design Peak Ground Acceleration
DEQ	=	Design Earthquake Magnitude
DSA	=	Design Spectral Acceleration
DSA_d	=	DSA values at damping other than 5%
DT	=	Displacement of vessel
DWT	=	Dead weight tons
d_{actual}	=	Arrival maximum draft of vessel at berth
d_{max}	=	Maximum vessel draft (in open seas)
E_{fender}	=	Energy to be absorbed by the fender system
E_{new}	=	% of extension at 100% breaking load for new rope
E_{vessel}	=	Berthing energy of vessel [ft-lbs]
F_a, F_v	=	Site coefficients from Tables 3-5 and 3-6
g	=	Acceleration due to gravity [32.2 ft/sec^2]
h	=	Elevation above water surface [feet]
K	=	Current velocity correction factor (Fig 3-4)
k	=	Radius of longitudinal gyration of the vessel [ft]
PGA_X	=	Peak ground acceleration corresponding to the Site Class under consideration.
s	=	Water depth measured from the surface [meters]
S_a	=	Spectral acceleration
S_1	=	Spectral acceleration value (for the boundary of S_B and S_C) at 1.0 second
S_A-S_F	=	Site classes as defined in Table 6-1
S_S	=	Spectral acceleration value (for the boundary of S_B and S_C) at 0.2
S_{X1}	=	Spectral acceleration value at 1.0 second corresponding to the Site Class under consideration
S_{XS}	=	Spectral acceleration value at 0.2 second corresponding to the period of S_S and the Site Class under consideration

T	=	Draft of vessel (see Fig 3-4)
T	=	Period (Sec)
T_o	=	S_{X1}/S_{XS}
V_c	=	Average current velocity [knots]
v_c	=	Current velocity as a function of depth [knots]
V_h	=	Wind speed (knots) at elevation h
V_L	=	Over land wind speed
V_n	=	Berthing velocity normal to the berth [ft/sec]
v_t	=	Velocity over a given time period
$V_{t=30\ sec}$	=	Wind speed for a 30 second interval
V_w	=	Wind speed at 33 ft. (10 m) elevation [knots]
W	=	Weight of the vessel in pounds [displacement tonnage x 2240]
WD	=	Water Depth (Fig 3-4)

3.12 REFERENCES

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4. SEISMIC ANALYSIS AND STRUCTURAL PERFORMANCE

4.1 GENERAL

4.1.1 Purpose

The purpose of Section 4 is to establish minimum standards for seismic analysis and structural performance. Two levels of seismic performance criteria at each MOT are evaluated. The Level 1 requirements define a performance level to ensure MOT functionality; Level 2 requirements safeguard against major structural damage or collapse.

4.1.2 Applicability

Section 4 applies to all new and existing MOTs. Structures supporting loading arms, pipelines and similar oil transfer and storage equipment or structures used to moor a vessel, such as mooring and breasting dolphins, need to be evaluated for seismic performance. Critical non-structural systems are also addressed. Catwalks and similar components that are not part of the lateral load carrying system and do not support oil transfer equipment may be excluded.

4.1.3 Recommended Practices

CalARP Program Seismic Guidance Committee, "Guidance for California Accidental Release Prevention (CalARP) Program Seismic Assessments," September 1998, Sacramento, CA.

Ferritto, J., Dickenson, S., Priestley N., Werner, S., Taylor, C., Burke D., Seelig W., and Kelly, S., 1999, "Seismic Criteria for California Marine Oil Terminals," Vol.1 and Vol.2, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, CA.

U.S. Department of Energy, March 1997, "Seismic Evaluation Procedures for Equipment at U.S. Department of Energy Facilities," DOE/EH-0545, Washington, D.C.

Working Group No. 34 of the Maritime Navigation Commission, 2001, "Seismic Design Guidelines for Port Structures," A. A. Balkema, Lisse, France.

4.1.4 Seismic Use Classification

Each existing MOT shall be classified into one of three risk categories shown in Table 4-1, based on the following three parameters:

- Exposed total volume of oil during transfer ("total volume" as calculated in Section 8.2.3)
- Number of oil transfer operations per berthing system per year
- Maximum vessel size (DWT) that may call at the berthing system

The low and moderate risk classification levels shall apply if either the number of transfers *or* vessel size requirements is met. If risk reduction strategies (See Section 1.4) are adopted, such that the maximum volume of exposed oil during transfer is less than 1,200 bbls, the classification level of the facility may be lowered. New MOTs are classified as high risk.

4.1.5 Configuration Classification

Each onshore MOT shall be designated as regular or irregular, in accordance with Figure 4-1.

Note that some irregular configurations, such as the "T" layout, may be analyzed as regular if the presence of expansion joints divides the T-configuration into two or more regular segments. Expansion joints in this context are defined as joints that separate each structural segment in such a manner that each segment will move independently during an earthquake.

TABLE 4-1
MOT RISK CLASSIFICATION

Risk Classification	Exposed Oil (bbls)	Transfers per Year per Berthing System	Maximum Vessel Size (DWTx1000)
High	≥1200	N.A.	N.A.
Moderate	<1200	≥90	≥30
Low	<1200	<90	<30

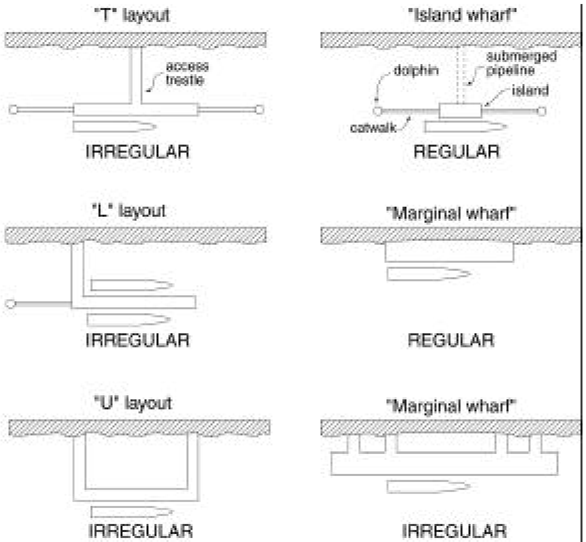


Figure 4-1: Pier and Wharf Configurations

If an irregular MOT is divided into regular segments which are seismically isolated, an evaluation of the relative movement of pipelines and supports shall be considered, including phase differences (See Section 9.3).

4.2 EXISTING STRUCTURES

4.2.1 Design Earthquake Motions

Two different levels of design earthquake motions for each MOT risk classification shall be considered. These levels are defined in Table 4-2.

The acceleration response spectrum (ARS) curves to be used for seismic assessment based on these design motions shall be established according to Section 3.4.

4.2.2 Performance Criteria

The criteria for seismic performance, as shown in Table 4-2 are:

Level 1 Seismic Performance:
Minor or no structural damage
Temporary or no interruption in operations

Level 2 Seismic Performance:
Controlled inelastic structural behavior with repairable damage
Prevention of structural collapse
Temporary loss of operations, restorable within months
Prevention of major spill (≥ 1200 bbls)

4.2.3 Basis for Evaluation

Structural and geotechnical information required for a seismic evaluation shall be obtained from drawings reflecting current as-built conditions, reports and codes/standards from the period of construction. If drawings are inadequate or unavailable, a Baseline Inspection shall be performed per Section 2.1.5.

Examination of selected components shall be conducted in accordance with Section 2.3.5.

Component capacities are based on current conditions, calculated as "best estimates" taking into account the mean material strengths, strain hardening and degradation over time. The capacity of components with little or no ductility, which may lead to brittle failure scenarios, shall be calculated based on lower bound material strengths.

Methods to establish component strength and deformation capacities are provided in Section 7 for typical structural materials and components.

TABLE 4-2
DESIGN EARTHQUAKE MOTIONS

Risk Classification	Seismic Performance Level	Probability of Exceedance	Return Period
High	Level 1	50% in 50 years	72 years
	Level 2	10% in 50 years	475 years
Moderate	Level 1	65% in 50 years	48 years
	Level 2	15% in 50% years	308 years
Low	Level 1	75% in 50 years	36 years
	Level 2	20% in 50 years	224 years

If adequate geotechnical data is not available, a site-specific investigation is required for areas subject to liquefaction, lateral spreading, slope instability, and for all MOTs in the "high" seismic risk classification. See Section 6 for geotechnical requirements.

4.2.4 Analytical Procedures

The objective of the structural seismic analysis is to verify that the displacement capacity of the structure is greater than the demand, for each performance level defined in Section 4.2.2.

The displacement capacity of the structure shall be calculated following the nonlinear static (pushover) procedure. The nonlinear dynamic (time-history) procedure may also be used.

Several methods can be used to calculate the displacement demand. The linear modal procedure is required for more complex structures but for simpler structures the nonlinear static procedure can be used to characterize the structural stiffness and thereby calculate the displacement demand. Again, the nonlinear dynamic procedure may be used.

The required analytical procedures are summarized in Table 4-3, and depend on the MOT risk classification, per Table 4-1. The methods are ranked in order of increasing complexity:

Capacity Procedures:
1. Nonlinear Static
2. Nonlinear Dynamic

Demand Procedures:
1. Linear Modal
2. Nonlinear Static
3. Nonlinear Dynamic

A more complex method can be used in lieu of the required analytical procedures shown in Table 4-3.

The displacement demand of pipelines relative to the structure shall be established to verify their elastic behavior; pipeline analysis is discussed in Section 4.5.2 and Section 9.3.

4.2.4.1 Nonlinear Static Capacity Procedure (Pushover)

Two-dimensional nonlinear static (pushover) analyses shall be performed for all MOTs; three-dimensional analyses are optional. A plan model that incorporates the nonlinear load deformation characteristics of all components for the lateral force-resisting system shall be displaced to a target displacement to determine the internal deformations and forces. The target displacement depends on the seismic performance level under consideration and may be calculated based on the methods shown in the following sections.

a) Modeling

A series of nonlinear pushover analyses may be required depending on complexity. At a minimum, pushover analysis of a two-dimensional model shall be conducted in both the longitudinal and transverse directions. In the model, the piles shall be represented

TABLE 4-3
MINIMUM REQUIRED ANALYTICAL PROCEDURES

Risk Classification	Configuration	Substructure Material	Demand Procedure	Capacity Procedure
High/Moderate	Irregular	Concrete/Steel	Linear Modal Procedure	Nonlinear Static Procedure
High/Moderate	Regular	Concrete/Steel	Nonlinear Static Procedure	Nonlinear Static Procedure
Low	Regular/Irregular	Concrete/Steel	Nonlinear Static Procedure	Nonlinear Static Procedure
High/Moderate/Low	Regular/Irregular	Timber	Nonlinear Static Procedure	Nonlinear Static Procedure

by nonlinear springs that capture the moment-curvature/rotation relationships for components with expected inelastic behavior in accordance with Section 7.

Linear material component behavior is acceptable where nonlinear response will not occur. All components shall be based on effective moment of inertia calculated in accordance with Section 7.

A nonlinear spring is not required to represent each pile location. Piles with similar lateral force-deflection behavior may be lumped in fewer larger springs provided that the overall torsional effects are captured.

b) Timber Pile Supported Structures

In general, timber pile deck connections may be assumed to be “pinned” (See Figure 4-2).

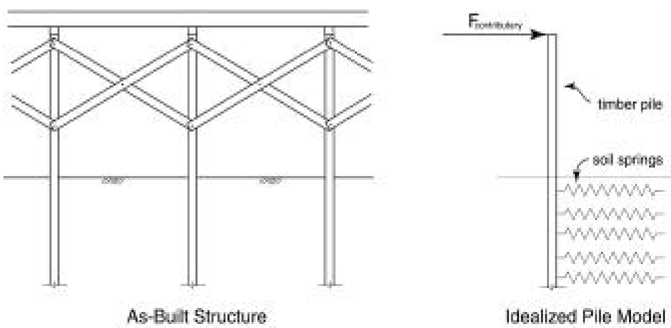


Figure 4-2
Timber Pile Supported Structure

In addition, the lateral bracing can often be ignored if in poor condition. These assumptions shall be used for the initial analysis, unless a detailed condition assessment and lateral analysis indicate that the existing bracing and connections provide reliable lateral resistance.

Given these assumptions, a series of single pile analyses may be sufficient to establish the nonlinear springs required for the pushover analysis.

c) Soil-Structure Interaction (SSI)

Load-deformation characteristics for foundations shall be modeled per Section 6. Selection of soil springs shall be based on the following assumptions:

- Effect of the large difference in up and down slope stiffness for wharf type structures
- Effect of upper and lower bound soil parameters, especially for t-z curves used to model batter pile behavior

A separate analysis that captures the demand (See Section 4.2.4.2) on the piles due to permanent ground deformations (at embankments only) shall be performed in accordance with either the simplified or advanced methodology outlined in Section 6.

If the simplified methodology is followed, the piles need to be checked for the following load combinations:

$$1.0E_{inertial} \quad (4.1)$$
$$1.0H_d + 0.25E_{inertial} \quad (4.2)$$

where:

$E_{inertial}$ = Inertial seismic load
H_d = Foundation deformation load

4.2.4.2 Nonlinear Static Demand Procedure

A nonlinear static procedure shall be used to determine the displacement demand for all concrete and steel structures, with the exception of irregular configurations with high or moderate seismic risk classifications. A linear modal procedure is required for irregular structures with high or moderate seismic risk classifications, and may be used for all other classifications in lieu of the nonlinear static procedure.

For all timber pile supported structures, the nonlinear static procedure may be used to estimate the target displacement demand, D_d.

a) Structural Period

The fundamental period (T) of the structure in the direction under consideration shall be calculated using the nonlinear force-displacement relation established by the pushover analysis. The period may be calculated as follows:

$$= 2p\sqrt{\frac{m}{k}} \quad (.3)$$

where:

= in kips/g
k stiffness in direction under consideration in kips/ft.

b) Lateral Stiffness

The lateral stiffness k is calculated from the force-displacement relation as the total base shear V_y corresponding to the yield displacement of the structure D_y. D_y is the displacement at first yield in the pile/deck connection reinforcement.

c) Target Displacement Demand

The target displacement demand of the structure D_d, can be calculated by multiplying the spectral response acceleration, S_A, corresponding to the period T, by $T^2/4\pi^2$

$$\Delta_d = S_A \frac{T^2}{4p^2} \tag{4.4}$$

If $T < T_o$, where T_o is the period corresponding to the peak of the acceleration response spectrum, a refined analysis (See Section 4.2.4.2(e)) shall be used to calculate the displacement demand. Multidirectional excitation shall be addressed per Section 4.4.2.

d) Damping

The displacement demand established in Section 4.2.4.2 (c) is based on 5% damping. Higher damping values obtained from a refined analysis may be used to calculate the displacement demand.

e) Refined Analyses

Refined displacement demand analyses may be calculated as follows [4.1]:

Establish D_d, from Section 4.2.4.2 (c). From the nonlinear pushover analysis, establish the structural yield displacement D_y. The ductility level m_D is found from D_d/D_y. Use the appropriate relationship between ductility and damping for the component undergoing inelastic deformation to estimate the effective structural damping, x_{eff}. In lieu of more detailed analysis, the relationship shown in Figure 4-2 or Equation 4.5 may be used for concrete and steel piles connected to the deck through dowels embedded in the concrete.

$$x_{eff} = 0.05 + \frac{1}{p}\left(1 - \frac{1-r}{\sqrt{m_\Delta}} - r\sqrt{m_\Delta}\right) \tag{4.5}$$

where:

r = ratio of second slope over elastic slope (see Figure 4-4)

From the acceleration response spectra, create elastic displacement spectra using equation 4.6 for various levels of damping. Using the curve most applicable to the effective structural damping, x, find the effective period, T_d (See Figure 4-3).

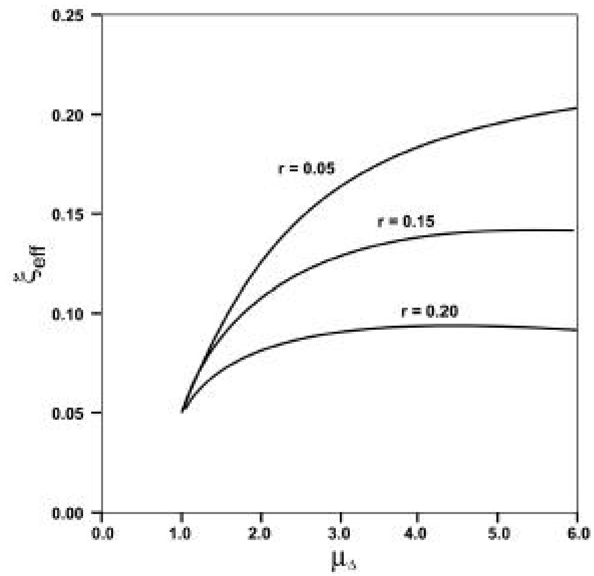


Figure 4-2: Relation Damping [4.1]

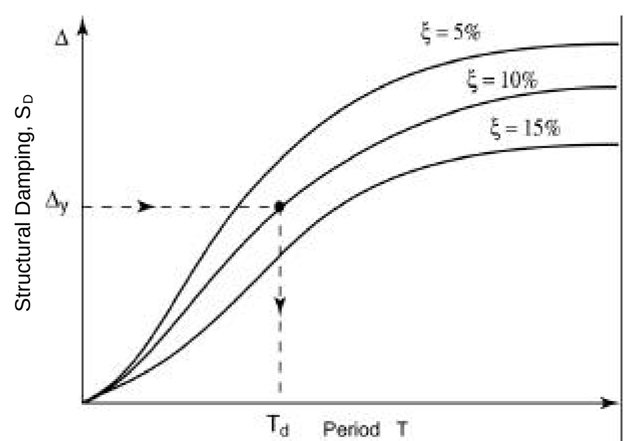


Figure 4-3 Spectra

$$S_D = \frac{^2}{4p^2} S_A \qquad (\ .6)$$

In order to convert from a design displacement response spectra to another spectra for a different damping level, the adjustment factors in Section 3.4.2.9 shall be used.

The effective stiffness k_e, can then be found from:

$$K_e = \frac{4p^2}{T_d^2} M \tag{4.7}$$

where:

M = mass of deck considered in the analysis.

The required strength F_u, can now be estimated by:

$$F_u = K_e D_d \tag{4.8}$$

F_u and D_d can be plotted on the force-displacement curve established by the pushover analysis. Since this is an iterative process, the intersection of F_u and D_d most likely will not fall on the force-displacement curve and a second iteration will be required. An adjusted value of D_d, taken as the intersection between the force-displacement curve and a line between the origin and F_u and D_d, can be used to find μ_Δ. Repeat the process until a satisfactory solution is obtained (See Figure 4-4).

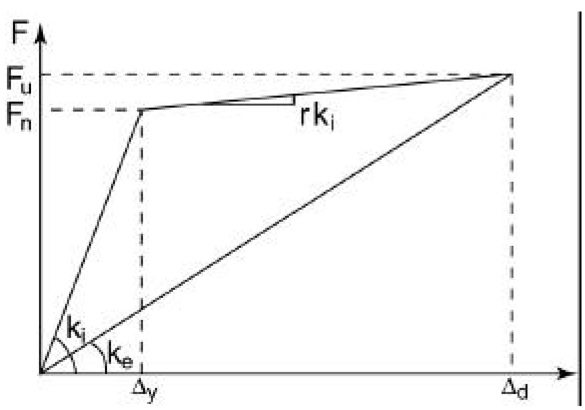


Figure 4-4 **e**

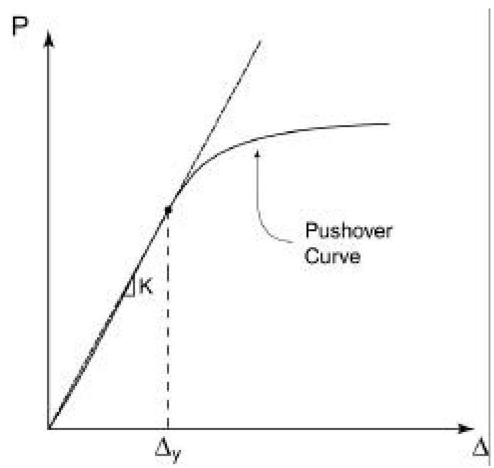
4.2.4.3 Demand Linear Modal Procedure

For all irregular structures with moderate or high seismic risk classifications, a linear elastic dynamic response spectra analysis is required to predict the global displacement demands of the MOT. A 3D linear elastic modal response analysis shall be used with effective moment of inertia applied to components to establish lateral displacement demands.

Sufficient modes shall be included in the analysis such that 90% of the participating mass is captured in each of the structures principal horizontal directions. For modal combinations, the Complete Quadratic Combination rule shall be used. Multidirectional excitation shall be accounted for in accordance to Section 4.4.2.

The lateral stiffness of the linear elastic response model shall be based on the initial stiffness of the nonlinear pushover curve as shown in Figure 4-5 (also see Sec. 6.6.1). The p-y springs shall be adjusted based on the secant method approach. Most of the p-y springs will typically be based on their initial stiffness; no iteration is required.

If the fundamental period in the direction under consideration is less than T_o, as defined in Section 4.2.4.2(c) then the displacement demand shall be amplified as specified in Section 4.2.4.2 (e).



: Stiffness for Linear Modal Analysis

Nonlinear Time History Analyses

performed, a peer review is required (See Section 1.6.2). Multiple acceleration records described in

The following assumptions may be made:

- Equivalent "super piles" can represent groups of piles.

  If the deck has sufficient rigidity to justify its

  out-of-plane, a 2-D plan simulation may be adequate.

compared with results from a simplified approach to ensure that results are reasonable. Displacements calculated from the nonlinear time history analyses may be used directly in design, but shall not be less than 80% of the values obtained from Sections 4.2.4.2.

4.2.4.5 Alternative Procedures

Alternative lateral-force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions. As per Section 1.6.2, peer review is required.

4.3 NEW STRUCTURES

All analysis and design requirements for existing MOTs described in Section 4.2 shall apply to new MOTs. All concrete or steel structures are classified as high seismic risk and require linear modal procedures. Additional requirements are:

- Site specific response spectra (See Section 3.4.2.3)
- Soil parameters based on site specific and new borings (See Section 6).

4.3.1 Design Earthquake Motions

There are two different levels of design earthquake motions for new facilities:

Level 1: Earthquake motion having a return period of 72 years corresponding to a 50% probability of occurrence in 50 years of exposure.

Level 2: Earthquake motion having a return period of 475 years corresponding to a 10% probability of occurrence in 50 years of exposure.

4.3.2 Performance Criteria

The seismic performance criteria are the same as defined in Section 4.2.2.

4.4 GENERAL ANALYSIS AND DESIGN REQUIREMENTS

4.4.1 Load Combinations

Earthquake loads shall be used in the load combinations described in Section 3.8.

4.4.2 Combination of Orthogonal Effects

The design displacement demand shall be calculated by combining the longitudinal and transverse displacements in the horizontal plane:

$$D_d = \sqrt{D_x^2 + D_y^2} \qquad (4.9)$$

where:

$$D_x = D_{xy} + 0\ 3D_{xx} \qquad (4$$

and

$$= 0.3D_{yx} + D_{yy} \qquad (4.11)$$

or

$$D_y = D_{yx} + 0.3D_{yy} \qquad (4.12)$$

and

$$D_x = 0.3D_{xy} + D_{xx} \qquad (4.13)$$

whichever results in the greater design displacement demand. Refer to Figure 4-6.

In lieu of combining the displacement demands as

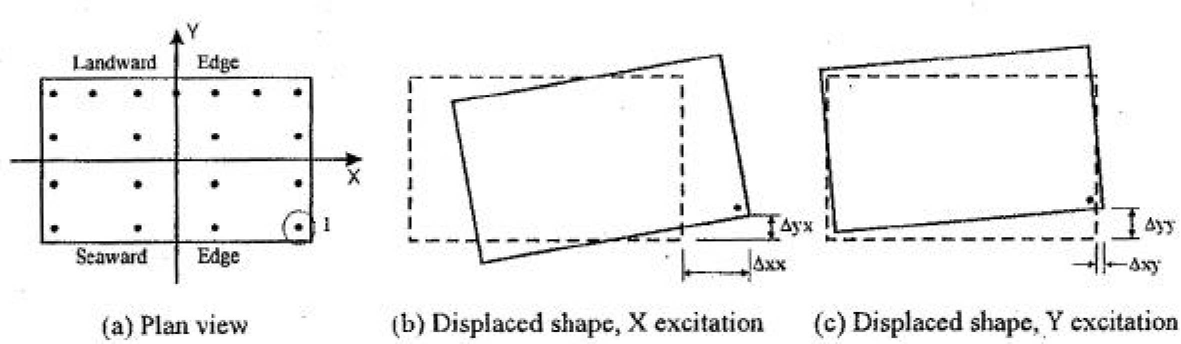


Figure 4-6 Plan View of Wharf Segment under X and Y seismic excitations

presented above, the design displacement demand for marginal wharf type MOTs may be calculated as:

$$D_d = D_y \sqrt{1 + (0.3(1 + 20e / L_l))^2} \qquad (4.14)$$

where:

D_y = transverse displacement demand

e = eccentricity between center of mass and center of rigidity

L_l = longitudinal length between wharf expansion joints

This equation is only valid for wharf aspect ratios (length/breadth) greater than 3.

4.4.3 P- Δ Effects

P-Δ effects shall be considered in seismic analysis; conversely, P-Δ effects may be ignored when the following relation is satisfied:

$$\frac{V}{W} \geq 4 \frac{D_d}{H} \qquad (4.15)$$

where:

V = base shear strength of the structure obtained from a plastic analysis

W = dead load of the frame

Δ_d = displacement demand

H = distance from maximum in-ground moment to center of gravity of the deck (see Figure 4-7)

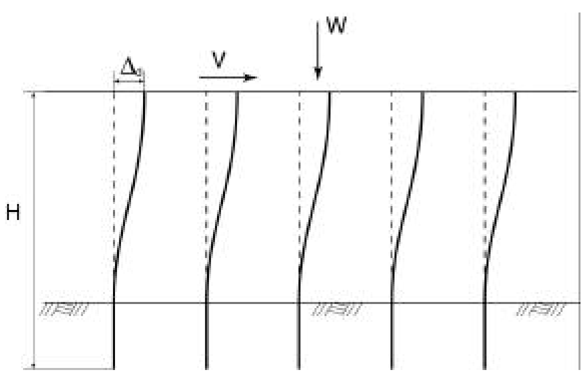


Figure 4-7: P-Δ Effect

For wharf structures where the lateral displacement is limited by almost fully embedded piles, P-Δ effects may also be ignored; however, the individual stability of the piles shall be checked in accordance with Section 7.3.5.2.

If the landside batter piles are allowed to fail in a level 2 evaluation, the remaining portion of the wharf shall be checked for P-Δ effects.

4.4.4 Expansion Joints

The effect of expansion joints shall be considered in the seismic analysis.

4.4.5 Shear Key Forces

Shear key forces shall be calculated according to Ferritto, et al (see 4.1.3).

4.4.6 Connections

For an existing wharf, the deteriorated conditions at the junction between the pile top and pile cap shall be considered in evaluating the moment capacity. Connection details between the vertical piles and pile caps shall be evaluated to determine whether full or partial moment capacity can be developed under seismic action. For new MOTs, the connection details shall develop the full moment capacities.

The modeling shall simulate the actual moment capacity (full or partial) of the joint in accordance with Section 7.3.7.

4.4.7 Batter Piles

Batter piles primarily respond to earthquakes by developing large axial compression or tension forces. Bending moments are generally of secondary importance. Failure may occur at the batter pile under either of the following conditions:

Compression:

- Failure of the deck/pile connection (most common type of failure)
- Material compression failure
- Buckling
- Excessive local shear in deck members adjacent to the batter pile.

Tension:

- Connection strength

- Pile dowel pull out, either by bond failure in the concrete or tensile failure in the dowels.

When the controlling failure scenario is reached and the batter pile fails, the computer model shall be adjusted to consist of only the vertical pile acting either full or partial moment frame based on the connection details between the pile top and pile cap. The remaining displacement capacity, involving vertical piles, before the secondary failure stage develops shall then be established (See Section 7.3.7.3).

Axial p-z curves shall be carefully modeled. In compression, displacement capacity should consider the effect of the reduction in pile modulus of elasticity at high loads and the increase in effective length for friction piles. This procedure allows the pile to deform axially before reaching ultimate loads, thereby increasing the displacement ductility (See Ferritto et al).

Horizontal nonlinear p-y springs are only applied to batter piles with significant embedment, such as for landside batter piles in a wharf structure. Moment fixity can be assumed for batter piles that extend well above the ground such as waterside batter piles in a wharf structure or batter piles in a pier type structure.

4.5 NONSTRUCTURAL COMPONENTS

Nonstructural components including, but not limited to pipelines, loading arms, raised platforms, control rooms and vapor control equipment may affect the global structural response. In such cases, the seismic characteristics (mass and/or stiffness) of the nonstructural components shall be considered in the structural analysis.

4.5.1 Mass Contribution

The weight of permanently attached nonstructural components shall be included in the dead load of the structure, per Section 3.2. An exception is an MOT pipeline that is allowed to slide between anchor points and hence the pipeline response is typically out of phase with the structural response. Thus, the pipeline may be subjected to a different acceleration than the substructure, even if the pipeline cannot slide between anchor points. In such cases, the pipeline mass shall not be included directly in the seismic mass of the structure.

4.5.2 Seismic Loads on Pipelines

Pipeline seismic analyses shall be performed in accordance with Section 9.3. Calculating seismic forces on a pipeline shall be based on established principles of mechanics. The evaluation procedures of FEMA 356 [4.2] Section 11.7 are adequate for MOT pipelines, and as a default, the following equations may be used:

$$F_p = 1.6 S_{ap} W_p I_p \qquad (4.16a)$$

$$F_{pv} = 0.667\, F_p \qquad (4.16b)$$

Where:

F_p = Seismic design force applied horizontally at the center of gravity of pipeline segment under consideration

F_{pv} = Seismic design force applied vertically to the center of gravity of pipeline segment under consideration

S_{ap} = Spectral response acceleration of pipeline segment under consideration

W_p = Weight of pipeline segment under consideration

I_p = Importance factor equal to 1.0

A pipeline segment under consideration shall extend between two adjacent anchor points. A simplified pipeline analysis may be used when the relative displacement demands of the anchor points are considered.

As an option, a full nonlinear time-history analysis can be used to capture the nonlinear interaction between the structure and the pipelines.

4.6 NONSTRUCTURAL CRITICAL SYSTEMS ASSESSMENT

A seismic assessment of the survivability and continued operation during a Level 2 earthquake (See Table 4-2) shall be performed for critical systems such as fire protection, emergency shutdown and electrical power systems. The assessment shall consider the adequacy and condition of anchorage, flexibility and seismically-induced interaction. The results shall be included in the Audit.

4.7 SYMBOLS

e = Eccentricity between center of mass and center of rigidity

$E_{inertial}$	=	Inertial seismic load
F_u	=	Required strength at maximum response
H	=	Distance from maximum in-ground moment to center of gravity of the deck
H_d	=	Foundation deformation load
k	=	Stiffness in direction under consideration in k/ft [kN/m]
K_e	=	Effective stiffness
L_l	=	Longitudinal length between wharf expansion joints
m	=	Mass of structure in kips/g [tonnes]
M	=	Mass of deck considered in the analysis
r	=	Ratio of second slope over elastic slope
S_A	=	Spectral response acceleration, at T
S_D	=	Displacement response spectrum, at T
T	=	Fundamental period of structure
T_d	=	Effective structural period
V	=	Base shear strength of the structure obtained from a plastic analysis
W	=	Dead load of the frame
D_d	=	Design displacement demand
D_x	=	Longitudinal displacement demand
D_{xx}	=	X displacement under X direction excitation
D_{xy}	=	X displacement under Y direction excitation
D_y	=	Transverse displacement demand
D_{yx}	=	Y displacement under X direction excitation
D_{yy}	=	Y displacement under Y direction excitation
m_D	=	Ductility level
x_{eff} *or* x	=	Effective structural damping

4.8 REFERENCES

[4.1] Priestley, M.J.N., Sieble, F., Calvi, G.M., 1996, "Seismic Design and Retrofit of Bridges," John Wiley & Sons, Inc., New York, USA.

[4.2] Federal Emergency Management Agency, FEMA-356, Nov. 2000, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," Washington, D.C.

5. MOORING AND BERTHING ANALYSIS AND DESIGN CRITERIA

5.1 GENERAL

5.1.1 Purpose

Section 5 establishes minimum standards for safe mooring and berthing of vessels at MOTs.

5.1.2 Applicability

Section 5 applies to onshore MOTs; Figure 5-1 shows typical pier and wharf configurations.

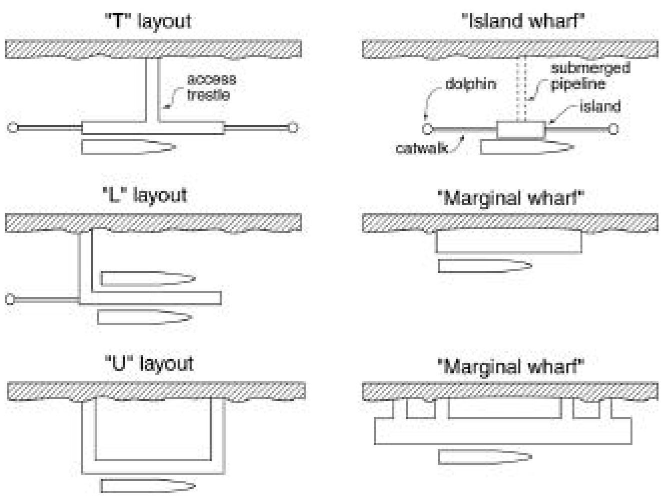


Figure 5-1: Typical Pier and Wharf Configurations

5.1.3 Classification of MOTs

Each MOT shall be assigned a mooring/berthing risk classification of high, medium or low, as determined from Table 5-1, based on the following site-specific environmental parameters:

- Wind
- Current
- Hydrodynamic effects of passing vessels
- Change in vessel draft

If any one of the four environmental conditions in Table 5-1 is met, then the corresponding highest risk classification shall be used for the MOT.

The maximum wind, V_w, (corrected for duration, height and over water) and maximum current, V_c, shall be obtained from Section 3.5. The requirement for passing vessel analysis is specified in Section 5.3.

Change in vessel draft shall be based on the local tidal variation and the operational limits of the vessels berthing at the MOT.

Multiple berth MOTs shall use the same environmental loads for each berth unless it can be demonstrated that there is a significant difference in the mooring demands.

MOTs classified as having high winds and/or high currents according to Table 5-1 shall have the following equipment in operation:

- Anemometer
- Current meter (may be omitted if safety factor according to Section 3.5.3.1 is applied to current)
- Remote reading tension load devices (new MOTs)

5.1.4 New MOTs

Quick release hooks are required at all new MOTs, except for spring line fittings. Quick release hooks shall be sized, within normal allowable stresses, for the safe working load of the largest size mooring line and configuration. To avoid accidental release, the freeing mechanism shall be activated by a two-step process. Quick release hooks shall be insulated electrically from the mooring structure, and should be supported so as not to contact the deck.

TABLE 5-1

MOT MOORING/BERTHING CLASSIFICATION

Risk Classification	Wind, (V_W) (knots)	Current, (V_c) (knots)	Passing Vessel Effects	Change in Draft (ft.)
High	>50	>1.5	Yes	>8
Moderate	30 to 50	1.0 to 1.5	No	6 to 8
Low	<30	<1.0	No	<6

5.1.5 MOT Characteristics

Structural characteristics of the MOT, including type and configuration of mooring fittings such as bollards, bitts, hooks and capstans and material properties and condition, shall be determined in accordance with Sections 4 and 7. In addition, the type and configuration of mooring lines, water depth and operational requirements, shall also be determined. The existing condition of the MOT shall be used in the mooring analysis.

5.1.6 Analysis and Design of Mooring Components

The analysis and design of mooring components shall be based on the loading combinations and safety factors defined in Sections 3.8 through 3.10, and in accordance with ACI 318, AISC-LRFD, ANSI/AF&PA NDS-2001, as referenced in Section 7.2, as applicable.

5.1.7 Applicable Codes and Recommended Practices

Mooring and berthing analysis and design criteria shall incorporate the guidelines and recommendations, as appropriate, from the following:

British Standards Institution, 1994, “British Standard Code of Practice for Maritime Structures - Part 4. Code of Practice for Design of Fendering and Mooring Systems”, BS6349, London, England.

Department of Defense, 1 July 1999, “Mooring Design,” Handbook, MIL-HDBK-1026/4A, Alexandria, VA, USA.

Department of the Navy, Dec. 1984, “Harbors Design Manual,” NAVFAC DM-26.1, Alexandria, VA, USA.

Department of the Navy, 30 October 1987, “Piers and Wharves,” Military Handbook, MIL-HDBK-1025/1, Alexandria, VA, USA.

Oil Companies International Marine Forum (OCIMF), 1997, “Mooring Equipment Guidelines”, 2nd Ed., London, England.

Oil Companies International Marine Forum (OCIMF), 1977, “Prediction of Wind and Current Loads on VLCCs,” London, England.

Seelig, William N., “Passing Ship Effects on Moored Ships”, 20 November 2001, Technical Report TR-6027-OCN, Naval Facilities Engineering Service Center (NFESC), Washington, D.C.

5.2 MOORING ANALYSES

A mooring analysis shall be performed for each berthing system, to justify the safe berthing of the various deadweight capacities of vessels expected at the MOT, in conformity with Section 2.3.6. The forces shall be determined in accordance with Section 3.5. Mooring line and breasting load combinations shall be in accordance with Sections 3.8.

A new mooring assessment shall be performed when conditions change, such as any modification in the mooring configuration, vessel size or new information indicating greater wind, current or other environmental loads.

In general, vessels shall remain in contact with the breasting or fendering system. Vessel motion (sway) of up to 2 feet off the breasting structure may be allowed under the most severe environmental loads, unless greater movement can be justified by an appropriate mooring analysis that accounts for potential dynamic effects. The allowable movement shall be consistent with mooring analysis results, indicating that forces in the mooring lines and their supports are within the allowable safety factors. Also, a check shall be made as to whether the movement is within the limitations of the cargo transfer equipment.

The most severe combination of the environmental loads has to be identified for each mooring component. At a minimum, the following conditions shall be considered:

- Two current directions (maximum ebb and flood; See Section 3.5)
- Two tide levels (highest high and lowest low)
- Two vessel loading conditions (ballast and maximum draft with sufficient underkeel clearance)
- Eight wind directions (45 degree increments)

Two procedures, manual and numerical are available for performing mooring analyses. These procedures shall conform to either the OCIMF documents, “Mooring Equipment Guidelines” and “Prediction of Wind and Current Loads on VLCCs” or the Department of Defense “Mooring Design” document (see 5.1.7).

5.2.1 Manual Procedure

For MOTs classified as *Low* risk (Table 5-1), simplified calculations may be used to determine the mooring forces, except if any of the following conditions exist (Figures 5-2 and 5-3, below).

- Mooring layout is significantly asymmetrical
- Horizontal mooring line angles (á) on bow and stern exceed 45 degrees
- Horizontal breast and spring mooring line angles exceed 15 and 10 degrees, respectively.
- Vertical mooring line angles (È) exceed 25 degrees
- Mooring lines for lateral loads not grouped at bow and stern

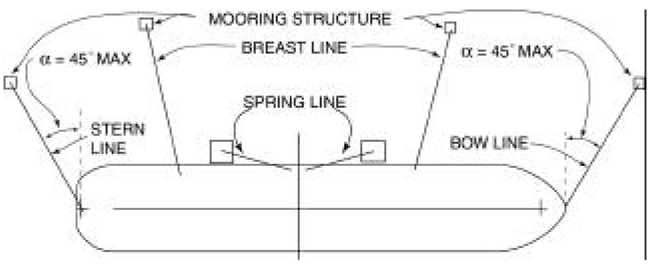


Figure 5-2: Horizontal Line Angles

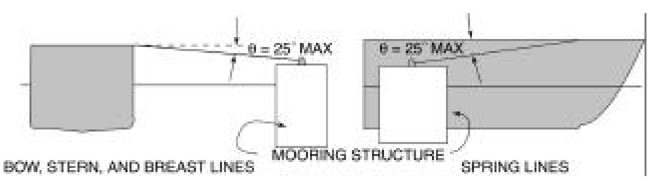


Figure 5-3: Vertical Line Angles

When the forces have been established and the distance between the bow and stern mooring points are known, the yaw moment can be resolved into lateral loads at the bow and stern. The total loads on a moored vessel can then be described by the following:

F_{yB} = Lateral load at vessel bow

F_{yS} = Lateral load at vessel stern

F_{xT} = Total longitudinal load

These loads are environmental loads; line pretension has to be added.

Four load cases shall be considered:

1. Entire load is taken by mooring lines
2. Entire load is taken by breasting structures
3. Load is taken by combination of mooring lines and breasting structures
4. Longitudinal load is taken only by spring lines

5.2.2 Numerical Procedure

A numerical procedure is required to obtain mooring forces for MOTs classified as *Moderate* or *High* (See Table 5-1) and for those that do not satisfy the requirements for using simplified calculations. The computer program(s) shall be based on acceptable mooring analysis procedures that consider the characteristics of the mooring system, calculate the environmental loads and provide resulting mooring line forces and vessel motions (surge and sway).

5.3 WAVE, PASSING VESSEL, SEICHE AND TSUNAMI CRITERIA

5.3.1 Wind Waves

MOTs are generally located in sheltered waters such that typical wind waves can be assumed not to affect the moored vessel if the significant wave period, T_s is less than 4 seconds. However, if the period is equal to or greater than 4 seconds, then a simplified dynamic analysis (See Section 3.5) is required. The wave period shall be established based on a 1-year significant wave height, H_s. For MOTs within a harbor basin, the wave period shall be based on the locally generated waves with relatively short fetch.

5.3.2 Passing Vessels

The forces generated by a passing vessel on a moored vessel may be significant. These forces are due to pressure gradients associated with the pattern of flow that accompanies the passing vessel. These pressure gradients cause the moored vessel to sway, surge, and yaw, thus imposing forces on the mooring lines.

Passing vessel analysis shall be conducted when all of the following conditions exist (See Figure 5-4):

- Passing vessel size is greater than 25,000 dwt.
- Distance L is 700 feet or less
- Vessel speed V is greater than V_{crit}

where:

$$V_{crit} = 1.5 + \frac{L - 2B}{700 - 2B} 4.5 \text{ (knots)} \tag{5.1}$$

Note: If L ≤ 2B, passing vessel loads shall be considered.

L and B are shown in Figure 5-4, in units of feet. V is defined as:

a) the speed of vessel over land minus the current velocity, when traveling with the current.
b) the speed of vessel over land plus the current velocity, when traveling against the current.

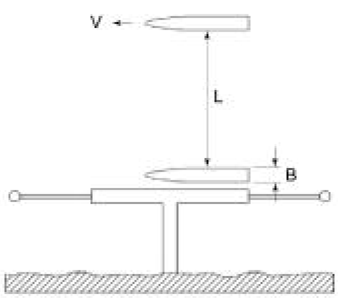


Figure 5-4: Passing Vessel

For MOTs located in ports, the passing distance may be established based on channel width and vessel traffic patterns. If such information is not available, the guidelines established in the Navy's "Harbors Design Manual," Figure 27, for interior channels may be used (see 5.1.7).

The "vertical bank" in Figure 27 shall be replaced by the side of the moored vessel when establishing the distance, "L". For MOTs, not located within a port, the distance must be determined from observed traffic patterns.

When such conditions exist, the surge and sway forces and the yaw moment acting on the moored vessel shall, as a minimum, be established in accordance with Section 3.5.5 as a screening process to evaluate the impact on the moored vessel. If the demands from such evaluation are greater than 75% of the mooring system capacity (breaking strength of mooring lines), then a more sophisticated dynamic analysis is required.

The mooring loads due to a passing vessel shall be added to the mooring loads due to wind and current.

The following passing vessel positions shall be investigated:

- Passing vessel is centered on the moored ship. This position produces maximum sway force.
- The mid-ship of the passing vessel is fore or aft of the centerline of the moored ship by a distance of 0.40 times the length of the moored ship. This position is assumed to produce maximum surge force and yaw moment at the same time.

5.3.3 Seiche

A seiche analysis is required for existing MOTs located within a harbor basin and which have historically experienced seiche. A seiche analysis is required for new MOTs inside a harbor basin prone to penetration of ocean waves.

The standing wave system or seiche is characterized by a series of "nodes" and "antinodes". Seiche typically has wave periods ranging from 20 seconds up to several hours, with wave heights in the range of 0.1 to 0.4 ft (see Navy's "Harbors Design Manual").

The following procedure may be used, as a minimum, in evaluating the effects of seiche within a harbor basin. In more complex cases where the assumptions below are not applicable, dynamic methods are required.

- Calculate the natural period of oscillation of the basin. The basin may be idealized as rectangular, closed or open at the seaward end. The formula provided in the Navy's "Harbor Design Manual", which calculates the wave period and length for different modes, shall be applied. The first three modes shall be considered in the analysis.
- Determine the location of the moored ship with respect to the antinode and node of the first three modes to determine the possibility of resonance.
- Determine the natural period of the vessel and mooring system. The calculation shall be based on the total mass of the system and the stiffness of the mooring lines in surge. The surge motion of the moored vessel is estimated by analyzing the vessel motion as a harmonically forced linear single degree of freedom spring mass system. Methods outlined in a paper by F.A. Kilner [5.1] can be used to calculate the vessel motion.
- Vessels are generally berthed parallel to the channel, therefore, only longitudinal (surge) motions shall be considered, with the associated mooring loads in the spring lines. The loads on the mooring lines (spring lines) are then determined from the computed vessel motion and the mooring line stiffnesses.

5.3.4 Tsunami

A tsunami generated by a distant source (far field event) may allow operators to have an adequate warning for mitigating the risk for departing the MOT and going into deep water. The MOT shall have a plan with specific actions for responding to tsunami events.

Table 3-8 provides run-up values for the San Francisco Bay area, Los Angeles/Long Beach Harbors and Port Hueneme.

5.4 BERTHING ANALYSIS AND DESIGN

The environmental loads used in the berthing analysis shall be established in accordance with Section 3.6. Existing MOTs shall consider both the fender system and structure for the berthing analysis, unless it can be demonstrated that the fender system alone can absorb the berthing energy.

The analysis and design of berthing components shall be based on the loading combinations and safety factors defined in Section 3.9 and in accordance with ACI 318, AISC-LRFD, ANSI/AF&PA NDS-2001, as referenced in Section 7.2, as applicable.

5.4.1 Berthing Energy Demand

The kinetic berthing energy demand shall be determined in accordance with Section 3.6.

5.4.2 Berthing Energy Capacity

The berthing energy capacity shall be calculated as the area underneath the force-deflection curve for the combined structure and fender system as indicated in Figure 5-5. The assumed contact length shall be in accordance with Section 5.4.3. Fender piles may be included in the lateral analysis to establish the total force-deflection curve for the berthing system. Load-deflection curves for other fender types shall be obtained from manufacturers, taking into account their current condition. The combined force-deflection curve can be established by adding the two curves together.

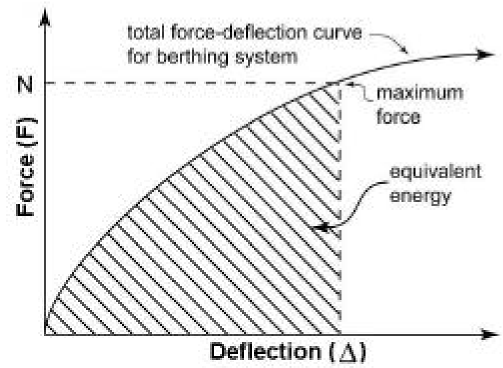


Figure 5-5: Berthing Energy Capacity

When batter piles are present, the fender typically absorbs most of the energy. This can be established by comparing the force-deflection curves for the two systems. In that case only the fender system energy absorption shall be considered.

5.4.3 Tanker Contact Length

5.4.3.1 Continuous Fender System

A continuous fender system consists of fender piles, chocks, wales, and rubber or spring fender units. The contact length of a ship during berthing depends on the spacing of the fender piles and fender units, and the connection details of the chocks and wales to the fender piles.

The contact length, L_c can be approximated by the chord formed by the curvature of the bow and the berthing angle as shown in Equation 5.2 below.

$$L_c = 2r \sin\alpha \qquad (5.2)$$

where:

L_c = contact length
r = Bow radius
a = Berthing Angle

In lieu of detailed analysis to determine the contact length, Table 5-2 may be used. The contact length for vessel sizes that are not listed in the table can be obtained by interpolation from the closest vessel sizes.

5.4.3.2 Discrete Fender System

For discrete fender systems (i.e. not continuous), one fender unit or breasting dolphin shall be able to absorb the entire berthing energy.

TABLE 5-2

CONTACT LENGTH

Vessel Size (dwt)	Contact Length
330	25 ft
1,000 to 2,500	35 ft
5,000 to 26,000	40 ft
35,000 to 50,000	50 ft
65,000	60 ft
100,000 to 125,000	70 ft

5.4.4 Longitudinal and Vertical Berthing Forces

The longitudinal and vertical component from the horizontal berthing force shall be calculated using appropriate coefficients of friction between the vessel and the fender. In lieu of as-built data, the values in Table 5-3 may be used for typical fender/vessel materials:

TABLE 5-3

COEFFICIENT OF FRICTION

Contact Materials	Friction Coefficient
Timber to Steel	0.4 to 0.6
Urethane to Steel	0.4 to 0.6
Steel to Steel	0.25
Rubber to Steel	0.6 to 0.7
UHMW* to Steel	0.1 to 0.2
*Ultra high molecular weight plastic rubbing strips	

Longitudinal and vertical forces shall be determined by:

$$F = m\,N \qquad (5.3)$$

where:

F = longitudinal or vertical component of horizontal berthing force (perpendicular to N-plane)

m = coefficient of friction of contact materials

N = maximum horizontal berthing force (normal to fender)

5.4.5 Design and Selection of New Fender Systems

For guidelines on new fender designs, refer to the Navy's "Piers and Wharves" handbook (5.1.7).

5.5 LAYOUT OF NEW MOTS

The number and spacing of independent mooring dolphins and breasting dolphins depends on the size and range of vessels to be accommodated.

Breasting dolphins shall be positioned within the parallel side of the vessel. They shall also be spaced far enough so that the berthing energy, thus the fender force, can be minimized. A minimum of two breasting dolphins shall be provided. Interior breasting dolphins may be required for the breasting of the smallest size vessel, if the difference in length between the maximum and minimum size vessel is large.

Mooring dolphins shall be set back from the berthing line (fender line) for a distance between 115 ft. and 165 ft. so that longer bow, stern and breast lines can be deployed.

For a preliminary layout, the guidelines in the British Standards, Part 4 (see 5.1.7), may be used in conjunction with the guidelines below.

- If four breasting dolphins are provided, the spacing between exterior breasting dolphins shall be approximately 0.25 x the LOA of the maximum sized vessel expected to call at the MOT. The spacing between interior breasting dolphins shall be approximately 0.40 x the LOA of the minimum sized vessel expected to call at the MOT.
- If only two breasting dolphins are provided, the spacing between the dolphins shall be the smaller (0.25 x LOA) of the guidelines specified above.
- If bow and stern lines are used for mooring, the spacing between exterior mooring dolphins shall be 1.35 times the LOA of the maximum sized vessel expected to call at the MOT.
- The spacing between interior mooring dolphins shall be 0.8 times the LOA of the maximum sized vessel expected to call at the MOT.

where:

LOA = Overall length of the vessel

The final layout of the mooring and breasting dolphins shall be determined based on the results of the mooring analysis that provides optimal mooring line and breasting forces for the range of vessels to be accommodated. The breasting force under the mooring condition shall not exceed the maximum fender reaction of the fender unit when it is being compressed at the manufacturers rated deflection.

5.6 SYMBOLS

á	=	Berthing Angle. It also means the angle of horizontal mooring lines, see Fig 5-2
B	=	Beam of vessel
F	=	Longitudinal or vertical component of horizontal berthing force
L	=	Distance between passing and moored vessels
L_c	=	Contact length
N	=	Maximum horizontal berthing force
r	=	Bow radius
m	=	Coefficient of friction of contact materials
V	=	Ground speed (knots)
V_c	=	Maximum current (knots).
V_{crit}	=	Ground speed (knots) above which passing loads must be considered
V_w	=	Maximum wind speed (knots)

5.7 REFERENCES

[5.1] Kilner F.A., 1961, “Model Tests on the Motion of Moored Ships Placed on Long Waves.” Proceedings of 7th Conference on COASTAL ENGINEERING, August 1960, The Hague, Netherlands, published by the Council on Wave Research - The Engineering Foundation.

6. GEOTECHNICAL HAZARDS AND FOUNDATIONS

6.1 GENERAL

6.1.1 Purpose

Section 6 provides minimum standards for analyses and evaluation of geotechnical hazards and foundations.

6.1.2 Applicability

The requirements provided herein apply to all new and existing MOTs.

6.1.3 Seismic Loading

The seismic loading for geotechnical hazard assessment and foundation analyses is provided in Section 3.4.

6.2 APPLICABLE RECOMMENDED PRACTICES

American Petroleum Institute, July 1993, Recommended Practice 2A-LRFD (API RP 2A-LRFD), "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design," Washington, D.C.

Federal Emergency Management Agency, FEMA-356, Nov. 2000, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," Washington, D.C.

Ferritto, J., Dickenson, S., Priestley N., Werner, S., Taylor, C., Burke D., Seelig W., and Kelly, S., 1999, "Seismic Criteria for California Marine Oil Terminals," Vol.1 and Vol.2, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, CA.

Southern California Earthquake Center (SCEC), March 1999, "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California," University of Southern California, Los Angeles.

6.3 SITE CHARACTERIZATION

6.3.1 Site Classes

Each MOT shall be assigned at least one site class, based on site-specific geotechnical information. Site Classes S_A, S_B, S_C, S_D, and S_E are defined in Table 6-1 and Site Class S_F is defined as follows:

1. Soils vulnerable to significant potential loss of stiffness, strength, and/or volume under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.

TABLE 6-1
SITE CLASSES

Site Class	Soil Profile Name/Generic Description	Average Values for Top 100 Feet of Soil Profile		
		Shear Wave Velocity, V_S [ft/sec]	Standard Penetration Test [blows/ft]	Undrained Shear Strength, S_U [psf]
S_A	Hard Rock	>5,000	-	-
S_B	Rock	2,500 to 5,000	-	-
S_C	Very Stiff/Dense Soil and Soft Rock	1,200 to 2,500	>50	>2,000
S_D	Stiff/Dense Soil Profile	600 to 1,200	15 to 50	1,000 to 2,000
S_E	Soft/Loose Soil Profile	<600	<15	<1,000
S_F	Defined in Section 6.3.1			

Note:
1. Site Class S_F shall require site-specific geotechnical information as discussed in Sections 6.3.2 and 3.4
2. Site Class S_E also includes any soil profile with more than 10 feet of soft clay defined as a soil with a plasticity index, PI>20, water content >40 percent and SU <500 psf.
3. The plasticity index, PI, and the moisture content shall be determined in accordance with ASTM D4318 [6.1] and ASTM D2216 [6.2], respectively.

2. Peats and/or highly organic clays, where the thickness of peat or highly organic clay exceeds 10 feet.
3. Very high plasticity clays with a plasticity index (PI) greater than 75, where depth of clay exceeds 25 feet.
4. Very thick soft/medium stiff clays, where the depth of clay exceeds 120 feet.

6.3.2 Site-Specific Information

In general, geotechnical characterization shall be based on site-specific information. This information may be obtained from existing or new sources. However, if existing or non-site specific information is used, the geotechnical engineer of record shall provide adequate justification for its use.

Site-specific investigations shall include, at a minimum, borings and/or cone penetration tests, soil classifications, configuration, foundation loading and an assessment of seismic hazards. The number and depths of exploratory borings and cone penetration tests shall be consistent with proposed or existing structures and site stratigraphy. The investigation or testing activities shall be completed following the procedures in SCEC (1999) (see Sec. 6.2).

When the geotechnical investigation is conducted to address liquefaction potential, the field investigation shall conform to Section 5.0 of SCEC (1999). The specific procedures to perform the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT) in Sections 5.4 and 5.5 of SCEC (1999) shall be followed. CPT data may also be used by first converting to SPT data, using an appropriate method, that reflects the effects of soil gradation. If geotechnical data other than SPT and CPT are used, an adequate explanation and rationale shall be provided.

Quantitative soil information is required to a depth of 100 feet below the mudline, for assigning a Site Class (see Table 6-1). When data to a depth of 100 feet is unavailable, other information such as geologic considerations may be used to determine the Site Class.

6.4 LIQUEFACTION

A liquefaction assessment shall address triggering (Section 6.4.1) and the resulting hazards, using residual shear strengths of liquefied soils (Section 6.4.2).

6.4.1 Triggering Assessment

Liquefaction triggering shall be expressed in terms of the factor of safety (SF):

$$SF = CRR/CSR \qquad (6.1)$$

Where:

CRR = Cyclic Resistance Ratio

CSR = The Cyclic Stress Ratio induced by Design Peak Ground Acceleration (DPGA) or other postulated shaking

CSR shall be evaluated using the simplified procedure in Section 6.4.1.1 or site-specific response analysis procedures in Section 6.4.1.2.

The CRR shall be determined from SCEC (1999) Figure 7.1. Whenever possible, it is preferable to use both the SPT and CPT data.

Shaking-induced shear strength reductions in liquefiable materials are associated with the generation and accumulation of excess pore water pressure. For cases in which the factor of safety against liquefaction is greater than 1.4, reductions of shear strength for the materials for post-earthquake conditions may be neglected. For cases in which liquefaction is expected (i.e., the factor of safety against liquefaction is 1.0 or less), reduction of the material shear strength to an residual undrained shear strength level shall be considered, as described in Section 6.4.2. For cases with factors of safety against liquefaction less than 1.4, but greater than 1.0, a strength value intermediate to the material's initial strength and residual undrained shear strength should be selected based on the level of residual excess pore water pressure expected to be generated by the ground shaking (e.g., Figure 10 of Seed and Harder, 1990) [6.3].

6.4.1.1 Simplified Procedure

The simplified procedure to evaluate liquefaction triggering shall follow SCEC (1999). Cyclic stress ratio (CSR) is used to define seismic loading, in terms of the Design Peak Ground Acceleration (DPGA) and Design Earthquake Magnitude (DEM). DPGA and DEM are addressed in Section 3.4.2. CSR is defined as:

$$CSR = 0.65\left(\frac{DPGA}{g}\right)\left(\frac{s_v}{s_v'}\right)r_d\left(\frac{1}{r_{MSF}}\right) \qquad (6.2)$$

where:

g	=	gravitational constant
s_v	=	the vertical total stress
s'_v	=	the vertical effective stress
r_d	=	a stress reduction factor
r_{MSF}	=	the magnitude scaling factor

For values of r_{MSF} and r_d, see SCEC (1999) Figures 7.2 and 7.3, respectively. To evaluate r_{MSF}, the DEM value associated with DPGA shall be used.

6.4.1.2 Site Specific Response Procedure

In lieu of the simplified procedure, either one-dimensional or two-dimensional site response analysis may be performed using the ground motion parameters discussed in Section 3.4. The computed cyclic stresses at various points within the pertinent soil layers shall be expressed as values of CSR.

6.4.1.3 Results

For cases where the SF is greater than 1.4, no reduction in shear strength is necessary. Where the SF is less than 1.4, stiffness or shear strength (or both) shall be reduced. The procedures for this reduction are described below.

6.4.2 Residual Strength

The residual undrained shear strength is estimated from SCEC (1999) Figure 7.7, based on "equivalent clean sand SPT blowcount". When necessary, a conservative extrapolation of the range in SCEC (1999) Figure 7.7 should be made. Under no circumstances, shall the residual shear strength be higher than the shear strength based on effective strength parameters.

The best estimate value should correspond to 1/3 from the lower bound of the range for a given value of equivalent clean sand SPT blowcount. When a value other than the "1/3 value" is selected for the residual shear strength, the selection shall be justified. An alternate is provided in Stark and Mesri [6.4]. The residual strength of liquefied soils may be obtained as a function of effective confining pressures if a justification is provided. The resulting residual shear strength shall be used as the post-earthquake shear strength of liquefied soils.

6.5 GEOTECHNICAL HAZARDS

For a SF less than 1.4, the potential for the following hazards shall be evaluated:

1. Flow slides
2. Slope movements
3. Lateral Spreading
4. Ground settlement and differential settlement
5. Other surface manifestations

These hazards shall be evaluated, using the residual shear strength described above (6.4.2).

6.5.1 Stability of Earth Structures

If a slope failure could affect the MOT, a stability analysis of slopes and earth retaining structures shall be performed. The analysis shall use limit equilibrium methods that satisfy all of the force and/or moment equilibrium conditions.

For slope failure, if the factor of safety is 1.2 or greater, the possibility of flow slides can be precluded. However, seismically induced ground movements shall be addressed.

For cases with the computed factor of safety less than 1.2 but greater than 1.0, seismically induced ground movements should be evaluated using ground movement methods described in Sections 6.5.2 and 6.5.3.

For cases with the factor of safety is less than 1.0, mitigation measures shall be implemented per Section 6.7.

6.5.2 Simplified Ground Movement Analysis

The seismically induced ground settlement may be estimated using SCEC (1999) Section 7.6. Surface manifestation of liquefaction may be evaluated using SCEC (1999) Section 7.7. The results of this analysis shall be evaluated to determine if mitigation measures are required.

Seismically induced deformation or displacement of slopes shall be evaluated using the Makdisi-Seed [6.5] simplified method as described below.

The stability analysis shall be used with the residual shear strengths of soils to estimate the yield acceleration coefficient (K_y) associated with the critical potential movement plane. In general, the Design Peak Ground Acceleration (DPGA) shall be used as K_{max} and Design Earthquake Magnitude (DEM) as the earthquake magnitude, M. These parameters shall be used together with the upper bound curves of Makdisi-Seed [6.5], to estimate the seismically induced ground movement along the critical plane.

The value of K_{max} may be different from the DPGA value to reflect the effects of amplification, incoherence, etc. When such adjustments are made in converting DPGA to K_{max}, a justification shall be provided. Linear interpolation using the upper bound curves in Makdisi-Seed [6.5] or Ferritto et al (1999), Figure 4-10 can be used to estimate the seismically induced ground movement for other earthquake magnitudes.

For screening purposes only, lateral spreading shall be evaluated, using the simplified equations in Youd et al. [6.6]. The total seismically induced ground displacement shall include all contributory directions.

When the resulting displacement is greater than 0.1 ft., the use of the Makdisi-Seed simplified method or other similar methods shall be used to estimate lateral spreading.

If the computed displacement from the simplified method is less than 0.5 feet, the effects can be neglected.

If the computed displacements using simplified methods are more than 0.5 feet, the use of a more detailed ground movement analysis (See Section 6.5.3) may be considered. If the final resulting displacement, regardless of the method used, remains greater than 0.5 feet, it shall be considered in the structural analysis.

6.5.3 Detailed Ground Movement Analysis

As an alternative to the simplified methods discussed above, a two-dimensional (2D) equivalent linear or nonlinear dynamic analysis of the MOT and/or slopes and earth retaining systems may be performed.

An equivalent linear analysis should be adequate when the stiffness and/or strength of the soils involved are likely to degrade by less than one-third, during seismic excitation of less than 0.5 g's. Appropriate time histories need to be obtained to calculate seismically induced displacement (See Section 3.4.2). Such analysis should account for the accumulating effects of displacement if double-integration of acceleration time histories is used. The seismic stresses or stress time histories from equivalent linear analysis may be used to estimate seismically induced deformation.

A nonlinear analysis should be used if the stiffness and/or strength of the soils involved are likely to degrade by more than one-third during seismic motion.

If the structure is included in the analysis, the ground motion directly affects the structural response. Otherwise, the uncoupled, calculated movement of the soil on the structure shall be evaluated.

6.6 SOIL STRUCTURE INTERACTION

6.6.1 Soil Parameters

Soil structure interaction (SSI) shall be addressed for the seismic evaluation of MOT structures. SSI may consist of linear or non-linear springs (and possibly dashpots) for various degrees of freedom, including horizontal, vertical, torsional, and rotational, as required by the structural analysis.

Springs for foundations shall be evaluated using the procedures in Chapter 4 of FEMA 356. The "p-y" curves, "t-z" curves, and tip load – displacement curves for piles (nonlinear springs for horizontal and vertical modes and nonlinear vertical springs for the pile tip, respectively) and deep foundations shall be evaluated using Section G of API RP 2A-LRFD including the consideration of pile group effects. Equivalent springs (and dashpots) representing the degrading properties of soils may be developed.

Where appropriate, alternative procedures can be used to develop these parameters and rationale shall be provided. One simplified method is presented in the Naval Design Manual 7.02 [6.7] and provides deflection and moment for an isolated pile, subject to a lateral load.

6.6.2 Shallow Foundations

Shallow foundations shall be assumed to move with the ground. Springs and dashpots may be evaluated following Gazetas [6.8].

6.6.3 Underground Structures

Buried flexible structures or buried portions of flexible structures including piles and pipelines shall be assumed to deform with estimated ground movement with depth.

The settling soils shall be assumed to apply shear forces to buried structures or buried portions of structures including deep foundations.

6.7 MITIGATION MEASURES AND ALTERNATIVES

If the hazards and consequences addressed in Sections 6.4 and 6.5 are beyond acceptable ranges, the following options shall be considered:

a) Perform a more sophisticated analysis
b) Modify the structure
c) Modify the foundation soil

Examples of possible measures to modify foundation soils are provided in Table 4-1 of Ferritto et al (1999).

6.8 REFERENCES

[6.1] American Society for Testing and Materials (ASTM), 2002, “D2Z16-98 Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass,” West Conshohocken, PA.

[6.2] American Society for Testing and Materials (ASTM), 2002, “D4310 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils,” West Conshohocken, PA.

[6.3] Seed, R.B. and Harder, C.F., 1999, SPT-Based Analysis of Cyclic Port Pressure Generation and Undrained Residual Strength, Proceedings of the H.B. Seed Memorial Symposium, Editor: J.M. Duncan, BiTech Publishers Ltd., v. 2, pp. 351-376.

[6.4] Stark, T.D., and Mesri, G., 1992, Undrained shear strength of liquefied sands for stability analysis, Journal of Geotechnical Engineering, American Society of Civil Engineers, v118, n11, pp 1727-1747.

[6.5] Makdisi, F.I. and Seed, H.B., “Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations”, ASCE Journal of the Geotechnical Engineering Division, Vol 104, No. 7, pp. 849-867

[6.6] Youd, T. L., Hansen, C. M., and Bartlett, S. F., “Revised MLR Equations for Predicting Lateral Spread Displacement” Proceedings of the 7th U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, 1999.”

[6.7] “Foundations and Earth Structures”, Design Manual 7.02, Chapter 5, 1986, Naval Facilities Engineering Command, Alexandria, VA.

[6.8] Gazetas, G., “Formulas and Charts for Impedances of Surface and Embedded Foundations”, Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 9, September, 1991.

7. STRUCTURAL ANALYSIS AND DESIGN OF COMPONENTS

7.1 GENERAL

7.1.1 Purpose

Section 7 establishes the minimum performance standards for structural components of MOTs. Evaluation procedures for seismic performance, strength and deformation characteristics of concrete, steel and timber components are prescribed herein. Analytical procedures for structural systems are presented in Section 4.

7.1.2 Applicability

Section 7 addresses MOTs constructed using the following structural components and elements:

- Reinforced concrete decks supported by batter and/or vertical concrete piles.
- Reinforced concrete decks supported by batter and/or vertical steel piles, including pipe piles filled with concrete.
- Reinforced concrete decks supported by batter and/or vertical timber piles.
- Timber decks supported by batter or vertical timber, concrete, or steel pipe piles.

7.2 APPLICABLE CODES, STANDARDS, AND RECOMMENDED PRACTICES

a) Applicable Codes and Standards

Structural analysis and design of components shall conform to the requirements of the most recent codes, standards and recommended practices listed below.

American Concrete Institute, ACI 318-02, 2002, “Building Code Requirements for Structural Concrete (318-02) and Commentary (318R-02),” Farmington Hills, Michigan.

American Forest & Paper Association, 2001, “National Design Specification for Wood Construction,” ANSI/AF&PA NDS-2001, Washington, D.C.

American Institute of Steel Construction (AISC), 2001, “Manual of Steel Construction, Load and Resistance Factor Design (LRFD),” Third Edition, Chicago, IL.

American Society of Civil Engineers, Jan. 2000, “Minimum Design Loads for Buildings and Other Structures,” ASCE 7-98, Revision of ANSI/ASCE 7-95, Reston, VA.

Federal Emergency Management Agency, FEMA-356, Nov. 2000, “Prestandard and Commentary for the Seismic Rehabilitation of Buildings,” Washington, D.C.

b) Recommended Practices

Department of Defense, 1988, MIL-HDBK-1025/6, “General Criteria for Waterfront Construction 1025/6,” 15 May 1988, Washington, D.C.

Ferritto, J., Dickenson, S., Priestley N., Werner, S., Taylor, C., Burke D., Seelig W., and Kelly, S., 1999, “Seismic Criteria for California Marine Oil Terminals, Vol.1 and Vol.2,” Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, CA.

Naval Facilities Engineering Command, 1986, “Foundations and Earth Structures,” Design Manual 7.02, Alexandria, VA.

Working Group No. 34 of the Maritime Navigation Commission, International Navigation Association (PIANC), 2001, “Seismic Design Guidelines for Port Structures,” A. A. Balkema, Lisse, France.

7.3 CONCRETE DECK WITH CONCRETE OR STEEL PILES

7.3.1 Component Strength

The following parameters shall be established in order to compute the component strength:

- Specified concrete compressive strengths
- Concrete and steel modulus of elasticity
- Yield and tensile strength of mild reinforcing and prestressed steel and corresponding strains
- Confinement steel strength and corresponding strains
- Embedment length
- Concrete cover
- Yield and tensile strength of structural steel
- Ductility

In addition, for existing components, the following conditions shall be considered:

- Environmental effects, such as reinforcing steel corrosion, concrete spalling and cracking, chemical attack
- Fire damage
- Past and current loading effects, including overload, fatigue or fracture
- Earthquake damage
- Discontinuous components
- Construction deficiencies

7.3.1.1 Material Properties

Material properties of existing components, not determined from testing procedures, and of new components, shall be established using the following methodology.

The strength of structural components shall be evaluated based on expected material properties, except for non-ductile component strength, which shall be evaluated based on design material properties. "Capacity Design" [7.1] ensures that the strength of protected locations and actions are greater than the maximum feasible demand, based on high estimates of plastic hinge flexural strength. In addition, a series of pushover analyses, using moment curvature characteristics of pile hinges, based on realistic upper bound estimates may be required. The following values shall be used (Ferritto et al, 1999, see 7.2):

Evaluation of the strength of non-ductile components (shear):

$$f'_c = 1.0\, f'_c \tag{7.1a}$$
$$f_y = 1.0\, f_y \tag{7.1b}$$
$$f_p = 1.0\, f_p \tag{7.1c}$$

Evaluation of the strength of other components (moment, axial):

$$f'_c = 1.3\, f'_c \tag{7.2a}$$
$$f_y = 1.1\, f_y \tag{7.2b}$$
$$f_p = 1.0\, f_p \tag{7.2c}$$

Evaluation of capacity protected members, such as pile caps and joints (maximum demand):

$$f'_c = 1.7\, f'_c \tag{7.3a}$$
$$f_y = 1.3\, f_y \tag{7.3b}$$
$$f_p = 1.1\, f_p \tag{7.3c}$$

where:
f'_c = Compressive strength of concrete
f_y = Yield strength of steel
f_p = Yield strength of prestress strands

Alternatively, if a moment-curvature analysis is performed that takes into account the strain hardening of the steel, the demands used to evaluate the capacity protected components may be estimated by multiplying the moment-curvature values by 1.25.

Based on a historical review of the building materials used in the twentieth century, guidelines for tensile and yield properties of concrete reinforcing bars and the compressive strength of structural concrete have been established (see Tables 6-1 to 6-3, FEMA 356, see 7.2). The values shown in these tables can be used as default properties, only if as-built information is not available and testing is not performed. The values in Tables 7-1 and 7-2 are subject to equations 7.1 through 7.3.

7.3.1.2 Knowledge Factor (k)

Knowledge factor, *k*, shall be applied on a component basis.

The following minimum information is required for a component strength assessment:

TABLE 7-1
COMPRESSIVE STRENGTH OF STRUCTURAL CONCRETE (PSI)[1]

Time Frame	Piling	Beams	Slabs
1900-1919	2,500-3,000	2,000-3,000	1,500-3,000
1920-1949	3,000-4,000	2,000-3,000	2,000-3,000
1950-1965	4,000-5,000	3,000-4,000	3,000-4,000
1966-present	5,000-6,000	3,000-5,000	3,000-5,000

1. Concrete strengths are likely to be highly variable for an older structure

TABLE 7-2

TENSILE AND YIELD PROPERTIES OF REINFORCING BARS FOR VARIOUS ASTM SPECIFICATIONS AND PERIODS

				Structual[1]	**Intermediate[1]**	**Hard[1]**			
			Grade	**33**	**40**	**50**	**60**	**70**	**75**
			Minimum Yield[2] (psi)	**33,000**	**40,000**	**50,000**	**60,000**	**70,000**	**75,000**
ASTM	**Steel Type**	**Year Range[3]**	**Minimum Tensile[2] (psi)**	**55,000**	**70,000**	**80,000**	**90,000**	**80,000**	**100,000**
A15	Billet	1911-1966		X	X	X			
A16	Rail[4]	1913-1966				X			
A61	Rail[4]	1963-1966					X		
A160	Axle	1936-1964		X	X	X			
A160	Axle	1965-1966		X	X	X	X		
A408	Billet	1957-1966		X	X	X			
A431	Billet	1959-1966							X
A432	Billet	1959-1966					X		
A615	Billet	1968-1972			X		X		X
A615	Billet	1974-1986			X		X		
A615	Billet	1987-1997			X		X		X
A616	Rail[4]	1968-1997				X	X		
A617	Axle	1968-1997			X		X		
A706	Low-Alloy[5]	1974-1997						X	
A955	Stainless	1996-1997			X		X		X

General Note: An entry "X" indicates that grade was available in those years.

Specific Notes:
1. The terms structural, intermediate, and hard became obsolete in 1968.
2. Actual yield and tensile strengths may exceed minimum values
3. Until about 1920, a variety of proprietary reinforcing steels were used. Yield strengths are likely to be in the range from 33,00 psi to 55,000 psi, but higher values are possible. Plain and twisted square bars were sometimes used between 1900 and 1949
4. Rail bars should be marked with the letter "R."
5. ASTM steel is marked with the letter "W"

- A set of "as-built" drawings and/or sketches, documenting both gravity and lateral systems (Section 2.1.5)
- A visual condition survey, for structural components including identification of the size, location and connections of these components
- In the absence of material properties, values from limited in-situ testing or conservative estimates of material properties (Table 7-1 and 7-2)
- Assessment of component conditions, from an in-situ evaluation, including any observable deterioration

The knowledge factor, k, is 1.0 when comprehensive knowledge and understanding of component configuration has been obtained. Comprehensive knowledge includes the minimum information above, and:

- Original construction records, including drawings and specifications, and any post-construction modification data, accurately depicting as-built conditions
- Detailed geotechnical information, based on recent test data, including risk of liquefaction, lateral spreading and slope stability

<u>Otherwise, the knowledge factor shall be 0.75.</u>

Further guidance on the determination of the appropriate k value can be found in Table 2-1 of FEMA 356.

7.3.2 Component Stiffness

An appropriate stiffness that takes into account the stress and deformation levels experienced by the component shall be selected. Nonlinear load-deformation relations shall be used to represent the component load-deformation response. However, in lieu of using nonlinear methods to establish the stiffness and moment curvature relation of structural components, the equations of Table 7-3 may be used to approximate the effective elastic stiffness, EI_{eff}, for lateral analyses (see 7.6 for definition of symbols).

TABLE 7-3

EFFECTIVE ELASTIC STIFFNESS

Concrete Component	EI_{eff}/EI_{gross}
Reinforced Pile	$0.3 + N/(f'_c A_{gross})$
Pile/Deck Dowel Connection[1]	$0.3 + N/(f'_c A_{gross})$
Prestressed Pile[1]	$0.6 < EI_{eff}/EI_{gross} < 0.75$
Steel Pile	1.0
Concrete w/ Steel Casing	$(E_s I_s + 0.25 E_c I_c)/E_s I_s + E_c I_c$
Deck	0.5

1 The pile/deck connection and prestressed pile may also be approximated as one member with an average stiffness of 0.42 EI_{eff}/EI_{gross} (Ferritto et al, 1999)
N is the axial load level.
E_S = Young's modulus for steel
I_S = Moment of inertia for steel section
E_C = Young's modulus for concrete
I_C = Moment of inertia for uncracked concrete section

7.3.3 Deformation Capacity of Flexural Members

Each structural component expected to undergo inelastic deformation shall be defined by its moment-curvature relation. The displacement demand and capacity shall be calculated per Section 4.2 and 4.3, as appropriate. Stress-strain models for confined and unconfined concrete, mild and prestressed steel presented in Section 7.3.4 shall be used to perform the moment-curvature analysis.

The stress-strain characteristics of steel piles shall be based on the actual steel properties. If as-built information is not available, the stress-strain relationship may be calculated per Section 7.3.4.2. For concrete in-filled steel piles, the stress-strain model for confined concrete shall be in accordance with Section 7.3.4.1.

The moment-rotation relationship for concrete components shall be derived from the moment-curvature analysis per Section 7.3.5.4.

A moment-rotation relationship for concrete components shall be used to determine the lateral displacement limitations of the design. Connection details shall be examined per Section 7.3.7.

7.3.4 Stress-Strain Models

7.3.4.1 Concrete

The stress-strain model and terms for both confined and unconfined concrete are shown in Fig. 7-1 (Priestley et al, Ref. [7.1]).

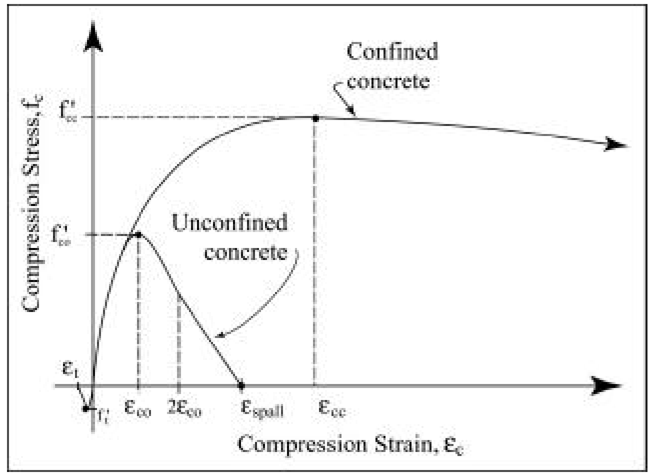


Figure 7-1: Stress-Strain Curves for Confined and Unconfined Concrete

7.3.4.2 Mild Reinforcement and Structural Steel

The stress-strain model and terms for reinforcing and structural steel are shown in Figure 7-2 (Priestley et al, Ref [7.1]).

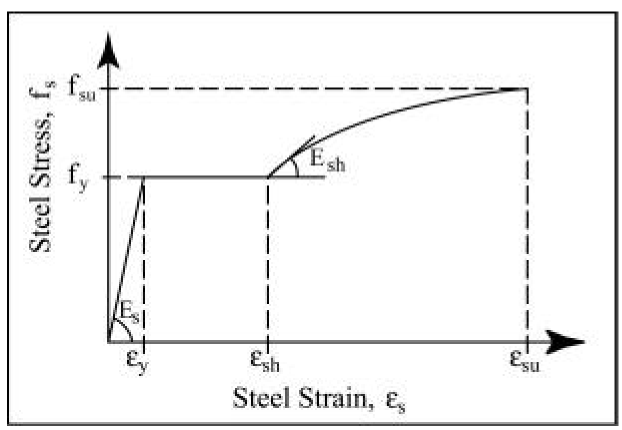


Figure 7-2 Stress-Strain Curve for Mild Reinforcing Steel or Structural Steel

7.3.4.3 Prestressed Steel

The stress-strain model of Blakeley and Park [7.2] may be used for prestressed steel. The model and terms are illustrated in Figure 7-3.

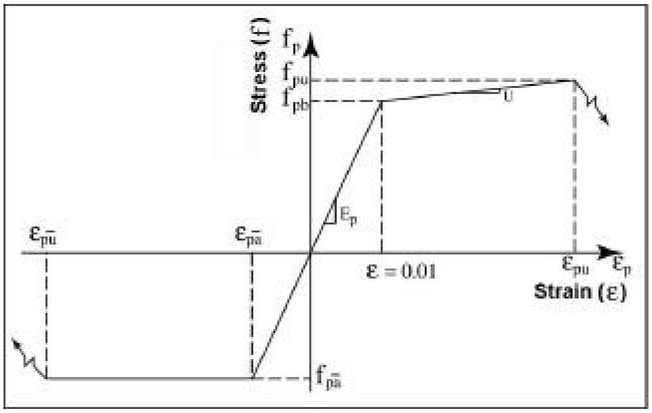


Figure 7-3: Stress-Strain Curve for Prestressed Steel

7.3.4.4 Alternative Stress-Strain Models

Alternative stress-strain models are acceptable if adequately documented and supported by test results.

7.3.5 Concrete Piles

7.3.5.1 General

The capacity of concrete piles is based on permissible concrete and steel strains corresponding to the desired performance criteria. Different values may apply for plastic hinges forming at in-ground and pile-top locations. Circular, octagonal, rectangular, and square piles are considered.

7.3.5.2 Stability

Stability considerations are important to pier-type structures. The moment-axial load interaction curve shall consider effects of high slenderness ratios (kl/r). The additional moment due to axial load eccentricity shall be incorporated unless:

$$e/h \leq 0.10 \tag{7.4}$$

where:
e = eccentricity of axial load
h = width of pile in considered direction

7.3.5.3 Plastic Hinge Length

The plastic hinge length needs to be determined to convert the moment-curvature relationship into a moment-plastic rotation relationship for the nonlinear pushover analysis.

The pile's plastic hinge length, L_p (above ground), when the plastic hinge forms against a supporting member is:

$$L_p = 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} \tag{7.5}$$

where:
L = the distance from the critical section of the plastic hinge to the point of contraflexure
d_{bl} = the diameter of the longitudinal reinforcement
f_{ye} = design yield strength of longitudinal reinforcement (ksi)

If a large reduction in moment capacity occurs due to spalling, then the plastic hinge length shall be:

$$L_p = 0.3 f_{ye} d_{bl} \tag{7.6}$$

When the plastic hinge forms in-ground, the plastic hinge length may be determined from Figure 7-4 (See Ferritto et al, 1999).

The stiffness parameter (x-axis) is:

$$\frac{KD^6}{[D^*]EI_{eff}} \tag{7.7}$$

where:
EI_{eff} = the effective stiffness
K = the subgrade modulus
D = shaft diameter
*D** = reference diameter of 6 ft

If more detailed information is not available then the values for *K* in Table 7-4 may be used.

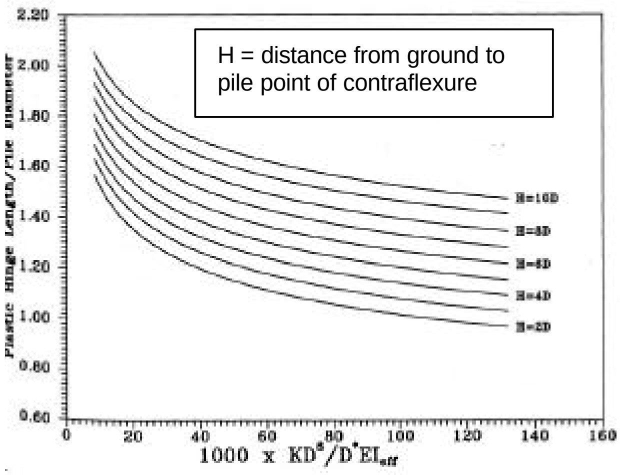


Figure 7-4: Influence of Pile/Soil Stiffness Ratio on Plastic Hinge Length

TABLE 7-4 SUBGRADE MODULUS *K*		
Soil Type	**Avg Undrained Shear Strength [psf]**	**Subgrade Modulus *K* [lb/in^3]**
Soft Clay	250-500	30
Medium Clay	500-1000	100
Stiff Clay	1000-2000	500
Very Stiff Clay	2000-4000	1000
Hard Clay	4000-8000	2000
Loose Sand (above WT/submerged)	-	25/20
Medium Sand (above WT/submerged)	-	90/60
Dense Sand (above WT/submerged)	-	275/125

7.3.5.4 Plastic Rotation

The plastic rotation can be determined from Equation 7-8, by using moment-curvature analysis and applicable strain limitations, as shown in Figure 7-5.

The plastic rotation is:

$$q_p = L_p f_p = L_p \left(f_m - f_y \right) \tag{7.8}$$

where:

L_p = plastic hinge length
f_p = plastic curvature
f_m = maximum curvature
f_y = yield curvature

The maximum curvature, f_m, shall be determined by the concrete or steel strain limit state at the prescribed performance level, whichever comes first.

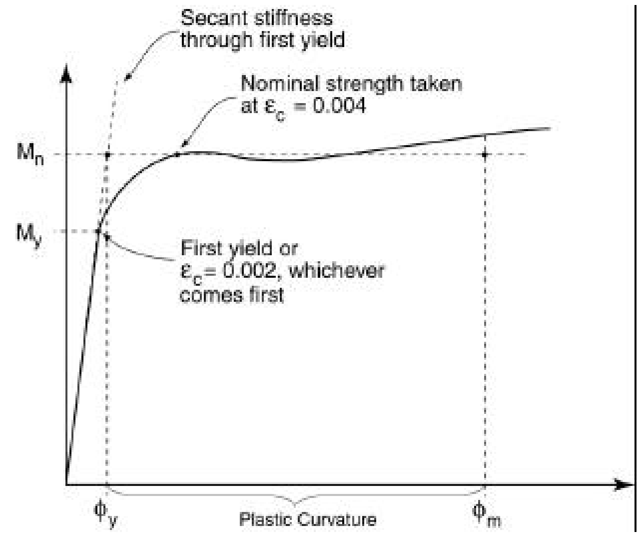


Figure 7-5: Moment Curvature Analysis

Alternatively, the maximum curvature may be calculated as:

$$f_m = \frac{e_{cm}}{c_u} \tag{7.9}$$

where:

e_{cm} = max extreme fiber compression strain according to 7.3.5.6.
c_u = neutral axis depth at ultimate strength of section

The yield curvature, f_y is the curvature at the intersection of the secant stiffness, EI_c , through first yield and the nominal strength, (e_c = 0.004)

$$f_y = \frac{M_y}{EI_c} \tag{7.10}$$

7.3.5.5 Ultimate Concrete and Steel Flexural Strains

Strain values computed in the nonlinear pushover analysis shall be compared to the following limits for flexure:

a) Unconfined concrete piles: An unconfined concrete pile is defined as a pile having no confinement steel or one in which the spacing of the confinement steel exceeds 12 inches.

Ultimate concrete compressive strain:

$$e_{cu} = 0.005 \qquad (7.11)$$

b) Confined concrete piles (See Ferritto et al), 1999:

Ultimate concrete compressive strain:

$$\varepsilon_{cu} = 0.004 + (1.4\rho_s f_{yh} \varepsilon_{sm})/f'_{cc} \geq 0.005 \qquad (7.12)$$
$$e_{cu,} \leq 0.035$$

where:

r_s = effective volume ratio of confining steel

f_{yh} = yield stress of confining steel

e_{sm} = strain at peak stress of confining reinforcement, 0.15 for grade 40, 0.12 for grade 60 and 0.10 for A82 grade 70 plain spiral

f'_{cc} = confined strength of concrete approximated by 1.5 f'_c

7.3.5.6 Component Acceptance/Damage Criteria

The maximum allowable concrete strains may not exceed the ultimate values defined in Section 7.3.5.5. The following limiting values apply for each performance level for both existing and new structures (see Table 7-5). The "Level 1 or 2" refer to the seismic performance criteria (See Section 4).

TABLE 7-5

LIMITS OF STRAIN

Component Strain	Level 1	Level 2
MCCS Pile/deck hinge	$\varepsilon_c \leq 0.005$	$\varepsilon_c \leq 0.025$
MCCS In-ground hinge	$\varepsilon_c \leq 0.005$	$\varepsilon_c \leq 0.008$
MRSTS	$\varepsilon_s \leq 0.01$	$\varepsilon_s \leq 0.05$
MPSTS In-ground hinge	$\varepsilon_p \leq 0.005$ (incremental)	$\varepsilon_p \leq 0.04$ (total strain)

MCCS = Maximum Concrete Compression Strain
MRSTS = Maximum Reinforcing Steel Tension Strain
MPSTS = Maximum Prestressing Steel Tension Strain

Concrete components for all non-seismic loading combinations shall be designed in accordance with ACI 318, as referenced in Section 7.2.

Note that for existing facilities, the pile/deck hinge may be controlled by the capacity of dowel reinforcement in accordance with Section 7.3.7.

7.3.5.7 Shear Capacity

Shear strength shall be based on nominal material strengths, and shear strength reduction factors.

To account for material strength uncertainties, maximum shear demand, $V_{max,push}$ established from nonlinear pushover analyses shall be multiplied by 1.4, from ATC-32 [7.3]:

$$V_{design} = 1.4V_{max,push} \qquad (7.13)$$

If moment curvature analysis that takes into account strain-hardening in accordance with Section 7.3.4 has been used to establish $V_{max,push}$ an uncertainty factor of 1.25 may be used (see ACI-318):

$$V_{design} = 1.25V_{max,push} \qquad (7.14)$$

If the factors defined in Section 7.3.1.1 are used, the above uncertainty factors need not be applied.

Kowalski and Priestley [7.4] may be used as an alternative to ACI-318. This method is based on a three-parameter model with separate contributions to shear strength from concrete (V_c), transverse reinforcement (V_s), and axial load (V_p) to obtain nominal shear strength (V_n):

$$V_n = V_c + V_s + V_p \qquad (7.15)$$

A shear strength reduction factor of 0.85 shall be applied to the nominal strength to determine the design shear strength. Therefore:

$$V_{design} \leq 0.85\ V_n \qquad (7.16)$$

The equations to determine V_c, V_s and V_p are:

a) The concrete mechanism shear strength, V_c

$$V_c = k\sqrt{f'_c}A_e \qquad (7.17)$$

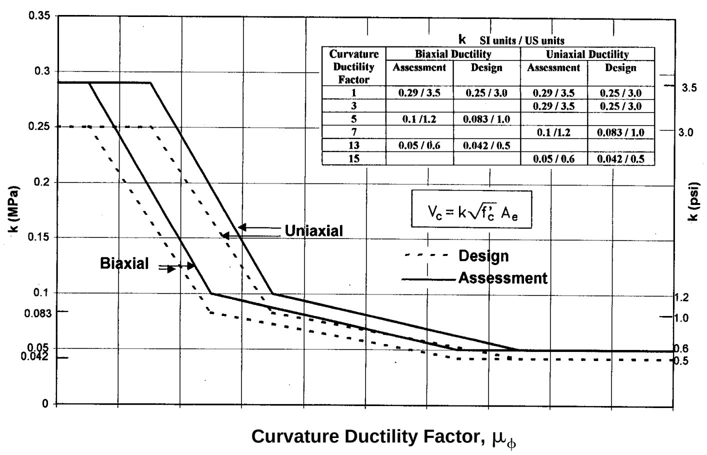
where:

k = factor dependent on the curvature ductility $\mu_\phi = \phi/\phi_y$, within the plastic hinge region, from Figure 7-6. For regions greater than $2D_p$ (see eqn. 7.18) from the plastic hinge location, the strength can be based on $\mu_\phi = 1.0$ (See Ferritto et al, Figure 3-30)

f'_c = concrete compressive strength

A_e = $0.8A_{gross}$ is the effective shear area

Figure 3-30. Concrete shear mechanism.



Curvature Ductility Factor	Biaxial Ductility Assessment	Biaxial Ductility Design	Uniaxial Ductility Assessment	Uniaxial Ductility Design
1	0.29 / 3.5	0.25 / 3.0	0.29 / 3.5	0.25 / 3.0
3			0.29 / 3.5	0.25 / 3.0
5	0.1 /1.2	0.083 / 1.0		
7			0.1 /1.2	0.083 / 1.0
13	0.05 / 0.6	0.042 / 0.5		
15			0.05 / 0.6	0.042 / 0.5

Figure 7-6: Concrete Shear Mechanism

b) Shear strength from Transverse reinforcement (truss) mechanism, V_s:

Circular spirals or hoops (Ref. [7.1]):

$$V_s = \frac{\frac{\pi}{2} A_{sp} f_{yh} \left(D_p - c - c_o\right) cot(\theta)}{s} \tag{7.18}$$

where:

A_{sp} = spiral or hoop cross section area

f_{yh} = yield strength of transverse or hoop reinforcement

D_p = pile diameter or gross depth (in case of a rectangular pile with spiral confinement)

c = depth from extreme compression fiber to neutral axis (N.A.) at flexural strength (see Fig. 7-7)

c_o = concrete cover to center of hoop or spiral (see Fig. 7-7)

θ = angle of critical crack to the pile axis (see Fig. 7-7) taken as 30° for existing structures, and 35° for new design

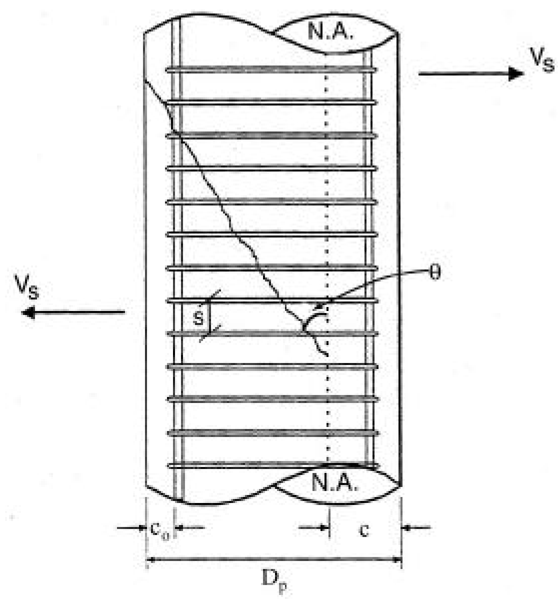


Figure 7-7: Transverse Shear Mechanism

s = spacing of hoops or spiral along the pile axis

Rectangular hoops or spirals (See Ref. [7.1]):

$$V_s = \frac{A_h f_{yh} \left(D_p - c - c_o\right) cot(\theta)}{s} \tag{7.19}$$

where:

A_h = total area of transverse reinforcement, parallel to direction of applied shear cut by an inclined shear crack

c) Shear strength from axial mechanism, V_p:

$$V_p = \Phi\left(N_u + F_p\right)\tan\alpha \tag{7.20}$$

where:

N_u = external axial compression on pile including seismic load. Compression is taken as positive; tension as negative.

F_p = prestress compressive force in pile

α = angle between line joining centers of flexural compression in the deck/pile and in-ground hinges, and the pile axis (see Fig. 7-8)

Φ = 1.0 for existing structures, and 0.85 for new design

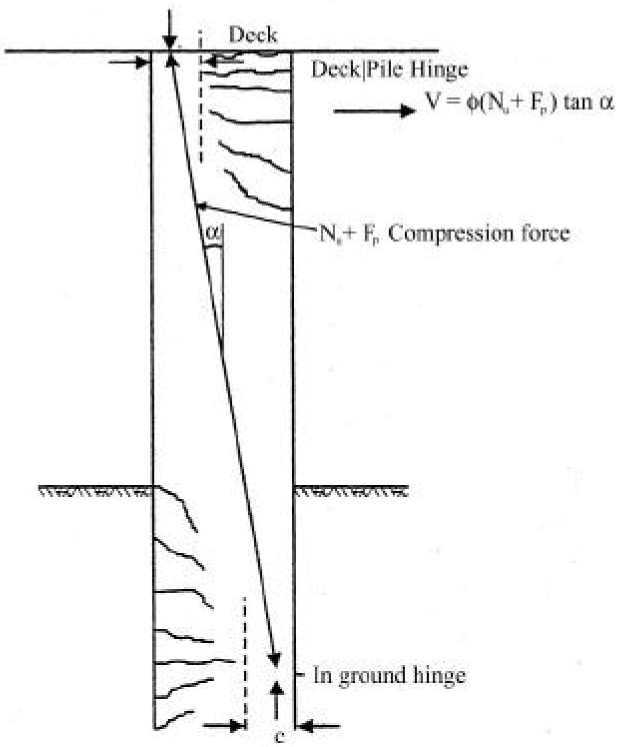


Figure 7-8: Axial Force Shear Mechanism

7.3.6 Steel Piles

7.3.6.1 General

The capacity of steel piles is based on allowable strains corresponding to the desired performance criteria and design earthquake.

7.3.6.2 Stability

The provisions in Section 7.3.5.2 shall also apply to steel piles.

7.3.6.3 Plastic Hinge Length

The plastic hinge length depends on the section shape and the slope of the moment diagram in the vicinity of the plastic hinge.

For plastic hinges forming in steel piles at the deck/pile interface and where the hinge forms in the steel section rather than in a special connection detail (such as a reinforced concrete dowel connection), allowance should be made for strain penetration into the pile cap. This increase may be taken as $0.25D_p$, where D_p is the pile diameter or pile depth in the direction of the applied shear force.

7.3.6.4 Ultimate Flexural Strain Capacity

The following limiting value applies:

Strain at extreme fiber, $\varepsilon_u \le 0.035$

7.3.6.5 Component Acceptance/Damage Criteria

The maximum allowable strain may not exceed the ultimate value defined in Section 7.3.6.4. Table 7-6 provides limiting values for each performance level, for both new and existing structures:

TABLE 7-6
STRUCTURAL STEEL STRAIN LIMITS, E_u

Component Strain	Level 1	Level 2
Concrete Filled Pipe	0.008	0.030
Hollow Pipe	0.008	0.025
Level 1 or 2 refer to the seismic performance criteria (Section 4)		

Steel and concrete components for all non-seismic loading combinations shall be designed in accordance with AISC-LRFD and ACI 318, as referenced in Section 7.2.

7.3.6.6 Shear Capacity

The procedures of Section 7.3.5.7 to establish V_{design} are applicable to steel piles (Equations 7.13 and 7.14). If the factors defined in Section 7.3.1.1 are used, the uncertainty factors need not be applied.

The shear capacity shall be established from the AISC-LRFD. For concrete filled pipe, the shear equations in Equation 7.15 may be used substituting V_{shell} with V_s:

$$V_{shell} = (\pi/2)tf_{yh}(D_p\text{-}c\text{-}c_0)\cot\theta \qquad (7.21)$$

where:
t = shell thickness
f_{yh} = yield strength of steel shell
c_o = outside of steel pipe to center of hoop or spiral
(All other terms are as listed for equation 7.18).

7.3.7 Pile/Deck Connection Strength

7.3.7.1 Joint Shear Capacity

The joint shear capacity shall be computed in accordance with ACI 318. For existing MOTs, the method [7.1, 7.5] given below may be used:

a) Determine the nominal shear stress in the joint region corresponding to the pile plastic moment capacity.

$$n_j = \frac{0.9M_p}{\sqrt{2}l_{dv}D_p^2} \quad (7.22)$$

where:

n_j = Nominal shear stress

M_p = Overstrength moment of the plastic hinge (the maximum possible moment in the pile) as determined from a pushover analysis at displacements corresponding to the damage control limit state (1.25 M_n when established from moment curvature and 1.3 and 1.1 over-strength factors are applied to f'_c and f_y, respectively, 1.4 otherwise.)

l_{dv} = Vertical development length, see Figure 7-9

D_p = Diameter of pile

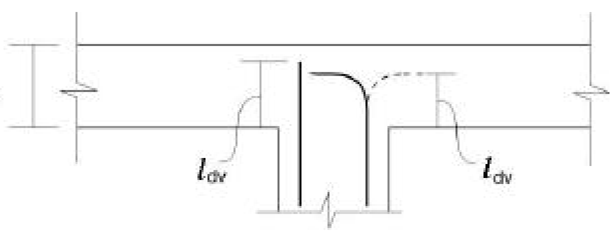


Figure 7-9: Development Length

b) Determine the nominal principal tension p_t, stress in the joint region:

$$p_t = \frac{-f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + n_j^2} \quad (7.23)$$

where:

$$f_a = \frac{N}{\left(D_p + h_d\right)^2} \quad (7.24)$$

is the average compressive stress at the joint center caused by the pile axial compressive force N and h_d is the deck depth. Note, if the pile is subjected to axial tension under seismic load, the value of N, and f_a will be negative.

If p_t, calculated above, exceeds $5.0\sqrt{f'_c}$ psi, joint failure will occur at a lower moment than the column plastic moment capacity M_p. In this case, the maximum moment that can be developed at the pile/deck interface will be limited by the joint principal tension stress capacity, which will continue to degrade as the joint rotation increases, as shown in Figure 7-10. The moment capacity of the connection at which joint failure initiates can be established from equations 7.26 and 7.27.

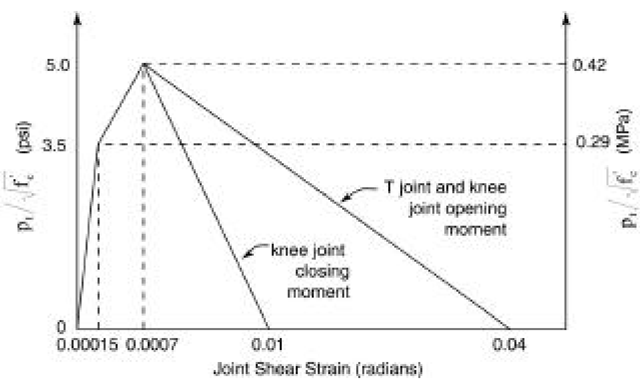


Figure 7-10: Degradation of Effective Principal Tension Strength with Joint Shear Strain (Ref. [7.1)

For $p_t = 5.0\sqrt{f'_c}$, determine the corresponding joint shear stress, n_j:

$$n_j = \sqrt{p_t(p_t - f_a)} \quad (7.25)$$

c) The moment capacity of the connection can be approximated as:

$$M_c = \sqrt{2}n_j l_{dv} D_p^2 1.1 \le M_p \quad (7.26)$$

This will result in a reduced strength and effective stiffness for the pile in a pushover analysis. The maximum displacement capacity of the pile should be based on a drift angle of 0.04 radians.

If no mechanisms are available to provide residual strength, the moment capacity will decrease to zero as the joint shear strain increases to 0.04 radians, as shown in Figure 7-11.

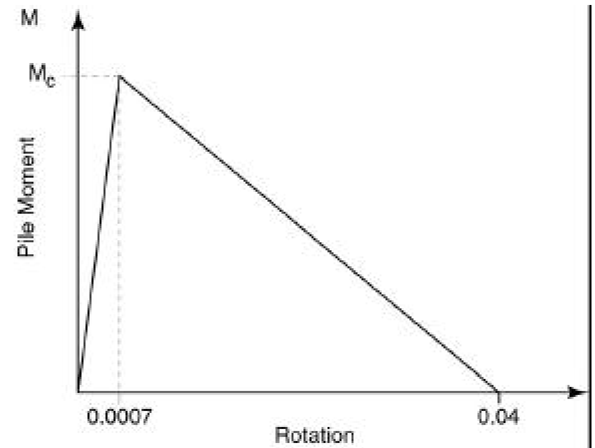


Figure 7-11 Reduced Pile Moment Capacity

If deck stirrups are present within $h_d/2$ of the face of the pile, the moment capacity, $M_{c,r}$, at the maximum plastic rotation of 0.04 radians may be increased from zero to the following (see Figure 7-12):

$$M_{c,r} = 2A_s f_y (h_d - d_c) + N(\frac{D_p}{2} - d_c) \quad (7.27)$$

As = Area of slab stirrups on one side of joint
h_d = See Figure 7-9 (slab thickness)
d_c = Depth from edge of concrete to center of main reinforcement (see eqn. 7.18)

In addition, the bottom deck steel area within $h_d/2$ of the face of the pile shall satisfy:

$$A_{s,bottomslab} \geq 0.5 \cdot A_s \quad (7.28)$$

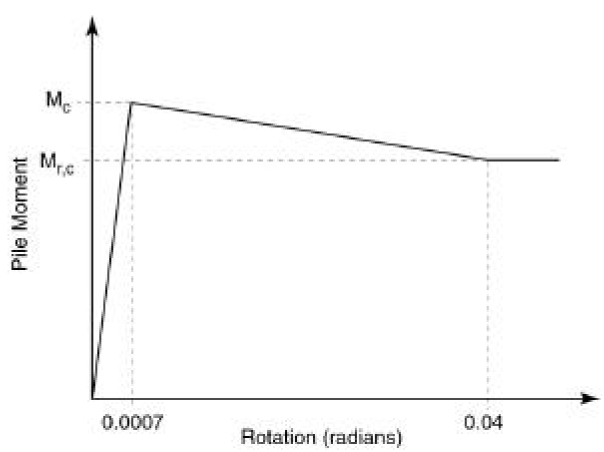


Figure 7-12: Joint Rotation

d) Assuming the same initial stiffness as before, the moment-curvature relationship established for the pile top can now be adjusted to account for the joint degradation.

The adjusted yield curvature can be found from:

$$f_y^{'} = \frac{f_y M_c}{M_n} \quad (7.29)$$

M_n is defined in Figure 7-5

The curvature corresponding to a joint rotation of 0.04 can be calculated as:

$$f_p = \frac{0.04}{L_p} \quad (7.30)$$

Where L_p is given by equation 7.5.

The adjusted ultimate curvature can now be calculated as:

$$f_u^{'} = f_p + \frac{f_y M_{c,r}}{M_n} \quad (7.31)$$

Note that $M_{c,r}$ = 0 unless slab stirrups are present as discussed above. Examples of adjusted moment curvature relationships are shown in Figure 7-13.

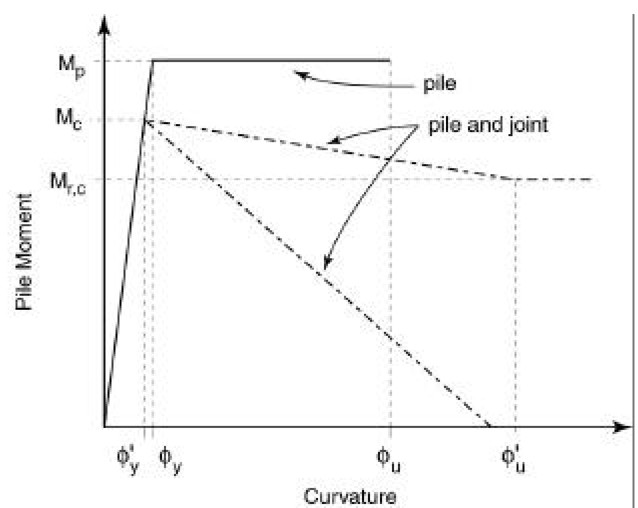


Figure 7-13 Equivalent Pile Curvature

7.3.7.2 Development Length

The required development length, l_{dc} , is:

$$l_{dc} \geq \frac{0.025 \cdot d_b \cdot f_{ye}}{\sqrt{f_c^{'}}} \tag{7.32}$$

where:

d_b = dowel bar diameter

f_{ye} = expected yield strength of dowel

In assessing existing details, actual or estimated values for f_{ye} and $f_c^{'}$ rather than nominal strength should be used in accordance with 7.3.1.1.

When the development length is less than that calculated by the equation 7.32, the moment capacity shall be calculated using a proportionately reduced yield strength, $f_{y,r}$, for the vertical pile reinforcement:

$$f_{y,r} = f_y \cdot \frac{l_d}{l_{dc}} \tag{7.33}$$

where:

l_d = actual development length

7.3.7.3 Batter Piles (Ordinary) for Existing MOTs

Wharves or piers with ordinary (not fused or having a seismic release) batter piles typically have a very stiff response when subjected to lateral loads in the direction of the batter. The structure often maintains most of its initial stiffness all the way to failure of the first row of batter piles. Since many batter piles most likely will fail under a level 2 seismic event, the following method may be used to evaluate the post batter pile failure behavior of the wharf or pier:

- Identify the failure mechanism of the batter pile/deck connection (refer to Section 4.4.7) for typical failure scenarios) and the corresponding lateral displacement.
- Release the lateral load between the batter pile and the deck when the lateral failure displacement is reached.
- Push on the structure until subsequent failure(s) have been identified.

Following these steps will result in a force-displacement (pushover) curve similar to the one shown in Figure 7-14 for a wharf supported by one row of batter piles.

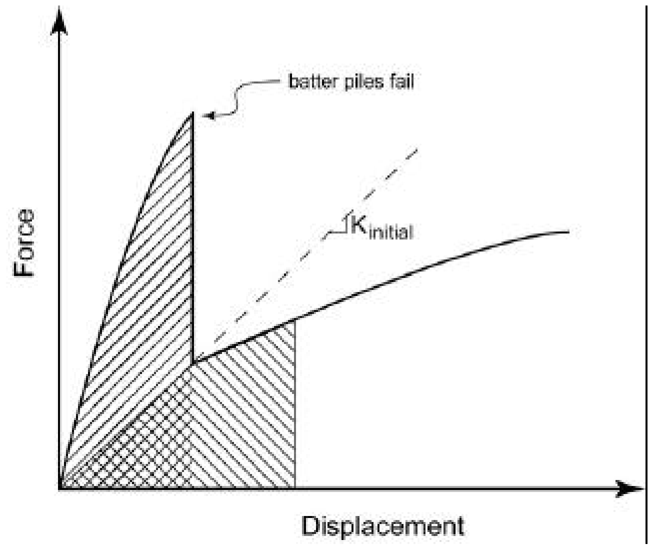


Figure 7-14: Pushover Curve for Ordinary Batter Piles

When the row of batter piles fail in tension/shear, stored energy will be released. The structure will therefore experience a lateral displacement demand following the non-ductile pile failures. If the structure can respond to this displacement demand without exceeding other structural limitations, it may be assumed that the structure is stable and will start to respond to further shaking with a much longer period and corresponding lower seismic demands. The wharf structure may therefore be able to sustain larger seismic demands following the loss of the batter piles than before the loss of pile capacity, due to the much softer seismic response.

The area under the pushover curve before the batter pile failures is compared to the equivalent area under the post failure pushover curve (refer to Figure 7-14). If no other structural limitations are reached with the new displacement demand, it is assumed that the structure is capable of absorbing the energy. It should be noted that even though the shear failure is non-ductile, it is expected that energy will be absorbed and the damping will increase during the damage of the piles. The above method is, therefore, considered conservative.

Following the shear failure of a pile row the period of the structure increases such that equal displacement can be assumed when estimating the post-failure displacement demand. The new period may be estimated from the initial stiffness of the post failure system as shown in Figure 7-14. A new displacement demand can then be calculated in accordance with Section 4.2.

7.3.8 Concrete Pile Caps with Concrete Deck

7.3.8.1 General

The moment-curvature and moment-rotation relationships shall be computed for pile caps using the same methodology as previously described. When the deck and the pile cap behave monolithically, an appropriate width of the deck may be included as part of the pile cap cross-section.

7.3.8.2 New and Existing MOTs

The pile cap and/or deck shall be designed to remain elastic for all design motions for new MOTs. For existing MOTs, the pile cap/deck shall remain elastic for Level 1 demands; however, inelastic action is allowed for Level 2 demands as long as the strain levels are in accordance Section 7.3.8.5.

7.3.8.3 Plastic Hinge Length

The plastic hinge length L_P, for existing pile caps may be taken as.

$$L_P = 0.5 \times D_c \qquad (7.34)$$

where D_c is the pile cap depth.

7.3.8.4 Ultimate Concrete and Steel Flexural Strains

The ultimate strain limits defined in Section 7.3.5.5 shall also apply to pile caps and deck.

All concrete shall be treated as unconfined concrete unless it can be demonstrated that adequate confinement steel is present.

7.3.8.5 Component Acceptance/Damage Criteria

The following limiting strain values apply for each performance level for new pile caps:

Deck/pile hinge: $\varepsilon_c \leq 0.005$ – Level 1&2
Reinforcing steel tension strain: $\varepsilon_s \leq 0.01$ – Level 1&2

The "Level 1 or 2" refer to the seismic performance criteria (See Section 4). For existing pile caps, the limiting strain values defined in Section 7.3.5.6 shall apply.

Concrete components for all non-seismic loading combinations shall be designed in accordance with ACI 318, as referenced in Section 7.2.

7.3.8.6 Shear Capacity

Shear strength shall be based on nominal material strengths, and shear strength reduction factors shall be employed in accordance with the latest version of ACI 318.

7.3.9 Concrete Detailing

For new MOTs, the required development splice length, cover and detailing shall conform to ACI 318, with the following exceptions:

a. For pile/deck dowels, the development length may be calculated in accordance with Section 7.3.7.2.
b. The minimum concrete cover for prestressed concrete piles shall be three inches, unless corrosion inhibitors are used, in which case a cover of two-and-one-half inches is acceptable.
c. The minimum concrete cover for wharf beams and slabs, and all concrete placed against soil shall be three inches, except for headed reinforcing bars (pile dowels or shear stirrups) the cover may be reduced to two and one-half inch cover at the top surface only. If corrosion inhibitors are used, a cover of two-and-one-half inches is acceptable.

7.4 TIMBER PILES AND DECK COMPONENTS

7.4.1 Component Strength

The following parameters shall be established in order to assess component strength:

New and existing components:

- Modulus of rupture
- Modulus of elasticity
- Type and grade of timber

Existing components only:

- Original cross-section shape and physical dimensions

- Location and dimension of braced frames
- Current physical condition of members including visible deformation
- Degradation may include environmental effects (e.g., decay, splitting, fire damage, biological and chemical attack) including its effect on the moment of inertia, I
- Loading and displacement effects (e.g., overload, damage from earthquakes, crushing and twisting)

Section 4.2.3 may be used to determine the existing material properties. At a minimum, the type and grade of wood shall be established. The published stress values in the ANSI/AF&PA NDS (7.2) may be used as default values and shall be multiplied by a factor of 2.8 to convert from allowable stress levels to yield or ultimate values for seismic loading.

For deck elements, the allowable stresses shall be limited to the values published in the ANSI/AF&PA NDS increased by a factor of 2.0. Piling deformation limits shall be calculated based on the strain limits in accordance with Section 7.4.3.

The values shown in the ANSI/AF&PA NDS are not developed specifically for MOTs and can be used as default properties only if as-built information is not available, the member is not damaged and testing is not performed. To account for the inherent uncertainty in establishing component capacities for existing structures with limited knowledge about the actual material properties, a reduction (knowledge) factor of k = 0.75 shall be included in the component strength and deformation capacity analyses in accordance with Section 7.3.1.2.

The modulus of elasticity shall be based on tests or the ANSI/AF&PA NDS. Alternatively the values shown in Table 7-5 may be used for typical timber piles:

TABLE 7-5

MODULUS OF ELASTICITY FOR TYPICAL TIMBER PILES

Species	E [psi]
Pacific Coast Douglas Fir	1,500,000
Red Oak	1,250,000
Red Pine	1,280,000
Southern Pine	1,500,000

7.4.2 Deformation Capacity of Flexural Members

The displacement demand and capacity of timber structures may be established per Section 4.2.

The soil spring requirements for the lateral pile analysis shall be in accordance with Section 6.

A linear curvature distribution may be assumed along the full length of a timber pile.

The displacement capacity of a timber pile can then be established per Section 7.4.3.3.

7.4.3 Timber Piles

7.4.3.1 General

The capacity of timber piles is based on allowable strains with consideration of both levels (1 and 2) of seismic performance.

7.4.3.2 Stability

The provisions in section 7.3.5.2 shall also apply to timber piles.

7.4.3.3 Displacement Capacity

A distinction shall be made between a pier type pile with a long unsupported length and a wharf landside type pile with a short unsupported length between the deck and soil. The effective length, L, is the distance between the pinned deck/pile connection and in-ground fixity as shown in Figure 7-15. For pier type (long unsupported length) vertical piles, two simplified procedures to determine fixity or displacement capacity may be found in the Department of Defense, MIL-HDBK-1025/6, “General Criteria for Waterfront Construction” or the Naval Design Manual 7.02 (See Section 7.2) respectively.

For soft soils, another alternative is to use Table 7-6 to determine fixity.

TABLE 7- 6

DISTANCE BELOW GROUND TO POINT OF FIXITY

Pile EI_{gross}	Soft Clays	Loose Granular & Medium Clays
< 10^{10} lb in^2	10 feet	8 feet
> 10^{10} lb in^2	12 feet	10 feet

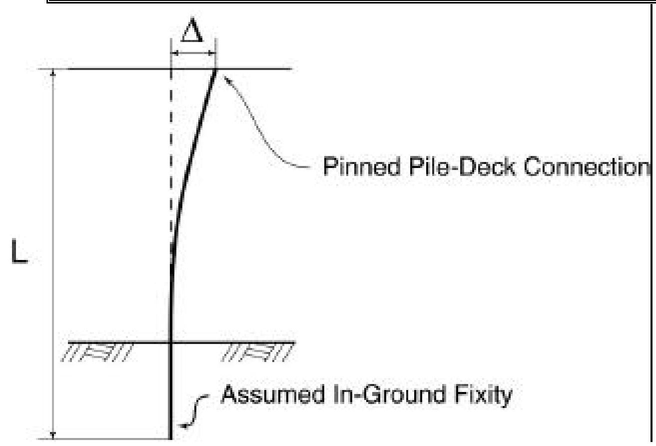


Figure 7-15: Assumed In-Ground Fixity

The displacement capacity, *D*, for a pile pinned at the top, with effective length, *L*, and moment, *M*, using Table 7-6 or MIL-HDBK-1025/6 is:

$$\Delta = \frac{ML^2}{3EI} \tag{7.35}$$

Assuming linear curvature distribution along the pile, the allowable curvature f_u for each load case can be established from:

$$f_u = \frac{e_a}{c_u} \tag{7.36}$$

where:

e_a = allowable strain limit according to 7.4.3.4

c_u = distance to neutral axis which can be taken as $D_p/2$

The curvature is also defined as:

$$f = \frac{M}{EI} \tag{7.37}$$

The maximum allowable moment therefore becomes:

$$M = \frac{2e_a}{D_p} EI \tag{7.38}$$

The displacement capacity is therefore given by:

$$D = \frac{2e_a L^2}{3D_p} \tag{7.39}$$

7.4.3.4 Component Acceptance/Damage Criteria

The following limiting values apply for each performance level for existing structures:

Earthquake Level	Max. Timber Strain
Level 1	0.004
Level 2	0.008

Timber components for all non-seismic loading combinations shall be designed in accordance with ANSI/AF&PA NDS (see Section 7.2).

7.4.3.5 Shear Capacity

Shear strength shall be based on nominal material strengths, and shear strength reduction factors shall be employed as outlined in this section.

To account for material strength uncertainties, the maximum shear demand, $V_{max,push}$ (see 7.3.5.7) established from the single pile lateral analysis shall be multiplied by 1.2:

$$V_{design} = 1.2V_{max,push} \tag{7.40}$$

The maximum shear stress T_{max}, in a circular pile can then be determined:

$$t_{\max} = \frac{10}{9}\frac{V_{\max,\,push}}{p \cdot r^2} \tag{7.41}$$

where:

r = radius of pile

The maximum allowable shear stress is the design shear strength from the ANSI/AF&PA NDS-1997 multiplied by a factor of 2.8 for the seismic load combination:

$$t_{capacity} = 2.8t_{design} \tag{7.42}$$

The shear capacity must be greater than the maximum demand.

7.5 MOORING AND BERTHING COMPONENTS

Section 7.5 applies to the following mooring and berthing components:

Mooring Components:

- Bitts
- Bollards
- Cleats
- Pelican Hooks
- Capstans
- Mooring dolphins
- Quick Release Hooks

Berthing components:

- Fender piles
- Fenders including camels, fender panels, and wales

Applicable safety factors to be applied to the demand are provided in Section 3.9.

7.5.1 Component Strength

The following parameters shall be established in order to calculate component strength:

New and existing components:

- Yield and tensile strength of structural steel
- Structural steel modulus of elasticity
- Yield and tensile strength of bolts
- Concrete infill compressive strength
- Concrete infill modulus of elasticity

For existing components (only):

- Condition of steel including corrosion
- Effective cross-sectional areas
- Condition of embedment material such as concrete slab or timber deck

Methods to determine the existing material properties are provided in Section 4.2.3.

7.5.2 Mooring and Berthing Component Demand

The maximum mooring line forces (demand) shall be established per Section 3.5. Multiple lines may be attached to the mooring component at varying horizontal and vertical angles. Mooring components shall therefore be checked for all the various mooring analysis load cases. The maximum demand on breasting dolphins and fender piles shall be established according to Section 3.6.

7.5.3 Capacity of Mooring and Berthing Components

The structural and connection capacity of mooring components bolted to the deck shall be established in accordance with AISC, ACI-318, ANSI/AF&PA NDS as appropriate. The mooring component capacity may be governed by the strength of the deck material. Therefore, a check of the deck capacity to withstand mooring component loads shall be performed.

7.6 SYMBOLS

A_e	=	$0.8A_{gross}$ is the effective shear area
A_{gross}	=	Uncracked, gross section area
A_h	=	Total area of transverse reinforcement, parallel to direction of applied shear cut by an inclined shear crack
A_{sp}	=	Spiral or hoop cross section area
c	=	Depth from extreme compression fiber to neutral axis at flexural strength (see Fig. 7-7)
c_o	=	Outside of steel pipe to center of hoop or spiral or concrete cover to center of hoop or spiral
c_u	=	Value of neutral axis depth at ultimate strength of section
D	=	Shaft diameter
D^*	=	Reference diameter of 6 ft
d_b	=	Dowel bar diameter
d_c	=	Depth from edge of concrete to center of reinforcement
d_{bl}	=	The diameter of the longitudinal reinforcement
D_c	=	Depth of pile cap
D_p	=	Pile diameter or gross depth (in case of a rectangular pile with spiral confinement)
e	=	Eccentricity of axial load
e_a	=	Allowable strain limit according to 7.4.3.4
e_{cm}	=	Max extreme fiber compression strain

according to 7.3.5.6.

e_{sm} = Strain at peak stress of confining reinforcement

EI_{eff} = The effective stiffness

EI_c The secant stiffness

EI_{gross} The gross stiffness of an uncracked concrete section, or gross stiffness of timber.

f'_c = Concrete compression strength in Mpa

f'_{cc} = Confined strength of concrete approximated by 1.5 f'_c

f_p = Stress in prestress strand

F_p = Prestress compression force in pile

f_{ye} = Design yield strength of longitudinal reinforcement (ksi)

f_{ye} = Expected yield strength of dowel

f_{yh} = Yield stress of confining steel

f_{yh} = Yield strength of transverse or hoop reinforcement

f_{yh} = Yield strength of steel shell

h = Width of pile in considered direction

H = Distance from ground to pile point of contraflexure

h_d = Depth of deck

K = The subgrade modulus

k = Factor dependent on the curvature ductility $m_f = f/f_y$,within the plastic hinge region, given by Figure 7-5.

L = The distance from the critical section of the plastic hinge to the point of contraflexure

l_d = Development length provided

l_{dv} = Vertical development length, see Figure 7-9

L_p = Plastic hinge length

M_p = As determined from a pushover analysis at displacements corresponding to the damage control limit state

N_u = External axial compression on pile including load due to earthquake action

r_s = Effective volume ratio of confining steel

r = Radius of circular pile

s = Spacing of hoops or spiral along the pile axis

t = Shell thickness

F = 1.0 for existing structures, and 0.85 for new design

q = Angle of critical crack to the pile axis (see Fig. 7-7) taken as 30° for existing structures, and 35° for new design

a = Angle between line joining centers of flexural compression in the deck/pile and in-ground hinges, and the pile axis (see Fig. 7-8)

f_m = Maximum curvature

f_y = Yield curvature

n_j = Nominal shear stress

7.7 REFERENCES

[7.1] M.J.N. Priestley, M.J.N, Seible, Frieder, Gian Michele Calvi, "Seismic Design and Retrofit of Bridges," 1996, New York.

[7.2] Blakeley, J.P., Park, R., "Prestressed Concrete Sections with Cyclic Flexure," Journal of the Structural Division, American Society of Civil Engineers, V. 99, No. ST8, August 1973, pp. 1717-1742, Reston, VA.

[7.3] Applied Technology Council, 1996, ATC-32, "Improved Seismic Design Criteria for California Bridges: Provisional Recommendations," Redwood City, CA,

[7.4] Kowalski, M.J. and Priestley, M.J.N., "Shear Strength of Ductile Bridge Columns," Proc. 5th Caltrans Seismic Design Workshop, Sacramento, June 1998.

[7.5] Priestley, M.J.N., facsimile message to CLSC, November 1st, 1999. See also "Seismic Design and Retrofit of Bridges."

8. FIRE PREVENTION, DETECTION, AND SUPPRESSION

8.1 GENERAL

8.1.1 Purpose

Section 8 provides minimum standards for Fire Prevention, Detection, and Suppression at Marine Oil Terminals (MOTs).

8.1.2 Applicability

The "new" (N) requirements of this section shall apply to:

- MOTs that commence or recommence operations after these standards are adopted
- Addition of new (non-replacement) equipment, components and systems to existing MOTs.

The "existing" (E) requirements of this section shall apply to all existing MOTs, equivalent or in-kind replacement of existing equipment, and minor modifications of components.

8.1.3 Applicable Codes, Standards, and Recommended Practices

a. Regulations, Codes and Standards

Fire prevention, detection and suppression systems shall conform to the requirements of Section 8, and the appropriate requirements of the following:

- 2 CCR 2300-2407 (Title 2, California Code of Regulations, Sections 2300-2407).
- 33 CFR (Title 33, Code of Federal Regulations) Part 154, Subpart C – Equipment Requirements and Section 154.550 – Emergency Shutdown.
- 49 CFR Part 195 – Transportation of Hazardous Liquids by Pipeline.
- American Petroleum Institute, 1994, "Design, Construction, Operation, Maintenance, and Inspection of Terminal and Tank Facilities," API Standard 2610, ANSI/API STD 2610-1994, 1st ed., Washington, D.C.
- National Fire Protection Association, 2002, NFPA 70, "National Electric Code," Quincy, MA.
- National Fire Protection Association, 1998, NFPA 10, "Portable Fire Extinguishers," Quincy, MA.
- National Fire Protection Association, 2000, NFPA 30, "Flammable and Combustible Liquids Code," Quincy, MA.

b. Recommended Practices

Fire prevention, detection and suppression systems shall incorporate the guidelines and recommendations, as appropriate, of the following publications:

- American Petroleum Institute, 1991, API Publication 2021, "Fighting Fires in and Around Flammable and Combustible Liquid Atmospheric Storage Tanks,", 3rd ed., Washington, D.C.
- American Petroleum Institute, 1999, API Publication 2218, "Fireproofing Practices in Petroleum and Petrochemical Processing Plants," 2nd ed., Washington, D.C.
- American Petroleum Institute, 1998, API Recommended Practice 2001 (API RP 2001), "Fire Protection in Refineries," 7th ed., Washington, D.C.
- American Petroleum Institute, 1998, API Recommended Practice 2003 (API RP 2003), "Protection Against Ignitions Arising Out of Static, Lightning, and Stray Currents," 6th ed., Washington, D.C.
- International Chamber of Shipping (ICS), Oil Companies International Marine Forum (OCIMF), International Association of Ports and Harbors (IAPH), 1996, "International Safety Guide for Oil Tankers and Terminals (ISGOTT)," 4th ed., Witherby, London.
- Oil Companies International Marine Forum (OCIMF), 1987, "Guide on Marine Terminal Fire Protection and Emergency Evacuation," 1st ed., Witherby, London.

8.2 HAZARD AND RISK EVALUATION

8.2.1 Fire Hazard Assessment and Risk Analysis (N/E)

A fire hazard assessment and risk analysis shall be performed, considering the loss of commercial power, earthquake and other relevant events. Examples of risk assessment methodologies are provided in [8.1] and [8.2].

8.2.2 Fire Plan (N/E)

A site-specific Fire Plan shall be prepared by a registered engineer or a competent fire protection professional. The plan shall consider the hazards and risks identified per Section 8.2.1 and shall include the elements of pre-fire planning as discussed in API RP 2021. The Fire Plan shall include goals, resources, organization, strategy and tactics, and also take into account the following:

a. MOT characteristics (e.g. tanker/manifold, product pipelines, etc.)
b. Product types and fire scenarios
c. Possible collateral fire damage, to adjacent facilities
d. Fire-fighting capabilities, including availability of water (flow rates and pressure), foam type and associated shelf life, proportioning equipment, and vehicular access
e. The selection of appropriate extinguishing agents
f. Calculation of water and foam capacities, as applicable, consistent with area coverage requirements
g. Coordination of emergency efforts
h. Emergency escape routes (2 CCR 2385 (d) (2))
i. Requirements for fire drills, training of wharf personnel, and the use of non-fixed equipment
j. Life safety, on and off-site
k. Rescue for terminal and vessel personnel
l. Cooling water for pipelines and valves exposed to the heat
m. Contingency planning when supplemental fire support is not available (Note: Mutual aid agreements can apply to water and land based support).
n. Consideration of adverse conditions, such as electrical power failure, steam failure, fire pump failure, an earthquake or other damage to the fire water system.

The audit team shall review and field verify the equipment locations, condition and operability as specified in the fire plan.

8.2.3 Hazard Classifications (N/E)

The cargo liquid hazard classes are defined in Table 8-1, as either High (H_C) or Low (L_C), depending on the flash point.

Fire hazard classifications (Low, Medium or High) are defined in Table 8-2, and are based on the cargo liquid hazard class and the sum of all *stored* and *flowing* volumes, prior to the Emergency Shut Down (ESD) stopping the flow of oil.

The *stored* volume is the sum of the H_C and L_C liquid hazard class piping volumes (V_{SH} and V_{SL}), if the piping is not stripped.

During the time prior to Emergency Shut Down (ESD), an additional volume of the cargo will be spilled. The ESD valve closure is required to be completed in 60 seconds if installed prior to November 1, 1980 or 30 seconds if installed after that date (2 CCR 2380 (h) (3)). The *flowing* volume is the sum of the H_C and L_C liquid hazard class volumes (V_{FH} and V_{FL}), and shall be calculated as follows:

$$V_F = Q_C \times t \times (1/3{,}600) \qquad (8.1)$$

where:

V_F = Flowing Volume (V_{FH} or V_{FL}) [bbl]
Q_C = Cargo Transfer Rate [bbl/hr]
t = ESD time, 30 or 60 seconds

Based on the total of the *stored* (S) and *flowing* (F) volumes ($V_{SL} + V_{SH} + V_{FL} + V_{FH}$), the MOT is classified as Low, Medium or High (Table 8-2).

TABLE 8-1
CARGO LIQUID HAZARD CLASS

Class	Criterion	Reference	Examples
Low (L_C)	Flash Point ≥ 140°F	**NFPA 30** – Combustible Class IIIA & IIIB **49 CFR 195** – Some Non-Flammable **ISGOTT** – Non-Volatile	#6 Heavy Fuel Oil, residuals, bunker
High (H_C)	Flash Point **<**140°F	**NFPA 30** – Combustible Class II & Flammable Class I **49 CFR 195** – Flammable & some Non-Flammable **ISGOTT** – Volatile	Gasoline, JP4, crude oils

TABLE 8-2

FIRE HAZARD CLASSIFICATIONS

Class	Stored Volume (bbl)			Flowing Volume (bbl)		Criteria (bbls)*
	Stripped	V_{SL}	V_{SH}	V_{FL}	V_{FH}	
LOW	y	y	y	y	y	$V_{FL} \geq V_{FH}$, & $V_T \leq 1200$
LOW	y	y	n	y	n	$V_{SL} + V_{FL} \leq 1200$
MEDIUM	y	n	y	n	y	$V_{SH} + V_{FH} \leq 1200$
MEDIUM	y	y	y	y	y	$V_{FH} > V_{FL}$, & $V_T \leq 1200$
MEDIUM	n	y	n	y	n	$V_{SL} + V_{FL} \leq 1200$
HIGH	y	n	y	n	y	$V_{SH} + V_{FH} > 1200$
HIGH	y	y	y	y	y	$V_T > 1200$
HIGH	n	y	y	y	y	$V_T > 1200$
HIGH	n	y	n	y	n	$V_{SL} + V_{FL} > 1200$
HIGH	n	n	y	n	y	$V_{SH} + V_{FH} > 1200$
HIGH	n	n	y	n	y	$V_{SH} + V_{FH} \leq 1200$

y ≡ yes
n ≡ no
Stripped ≡ product purged from pipeline following product transfer event.
V_{SL} ≡ stored volume of low hazard class product
V_{SH} ≡ stored volume of high hazard class product
V_{FL} ≡ volume of low hazard class product flowing through transfer line during 30 - 60 secs. ESD.
V_{FH} ≡ volume of high hazard class product flowing through transfer line during 30 - 60 secs. ESD.
$V_T \equiv V_{SL} + V_{SH} + V_{FL} + V_{FH}$ = Total Volume (stored and flowing)
* Quantities are based on maximum flow rate, including simultaneous transfers.

8.3 FIRE PREVENTION

8.3.1 Ignition Source Control

a. Protection from ignition by static electricity, lightning or stray currents shall be in accordance with API RP 2003 (N/E).

b. Requirements to prevent electrical arcing shall be in conformity with 2 CCR 2341 (N/E).

c. Multi-berth terminal piers shall be constructed so as to provide a minimum of 100 ft between adjacent manifolds (N).

8.3.2 Emergency Shutdown Systems

An essential measure of fire prevention is communications in conjunction with the emergency shutdown. The ESD and isolation system shall conform to 2 CCR 2380 (h) and 33 CFR Part 154. An ESD system shall include or provide:

a. An ESD valve, located near the dock manifold connection or loading arm (N/E).

b. ESD valves, with "Local" and "Remote" actuation capabilities (N).

c. Remote actuation stations strategically located, so that ESD valve(s) may be shut within required times (N).

d. Multiple actuation stations installed at strategic locations, so that one such station is located more than 100 feet from areas classified as NFPA 70 Class I, Group D, Division 1 or 2. Actuation stations shall be wired in parallel to achieve redundancy and arranged so that fire damage to one station will not disable the ESD system (N).

e. Communications or control circuits to synchronize simultaneous closure of the Shore Isolation Valves (SIVs) with the shut down of loading pumps (N).

f. A manual reset to restore the ESD system to an operational state after each initiation (N).

g. An alarm to indicate failure of the primary power source (N).

h. A secondary (emergency) power source (N).

i. Periodic testing of the system (N).

j. Fire proofing of motors and control-cables that are installed in areas classified as NFPA 70 Class I, Group D, Division 1 or 2. Fire proofing shall, at a minimum, comply with the requirements of API Publication 2218 (N).

8.3.3 Shore Isolation Valves (SIV)

Shore Isolation Valve(s) shall:

a. Be located onshore for each cargo pipeline. All SIVs shall be clustered together, for easy access (N).
b. Be clearly identified together with associated pipeline (N/E).
c. Have adequate lighting (N/E).
d. Be provided with communications or control circuits to synchronize simultaneous closure of the ESD with the shut down of loading pumps (N).
e. Have a manual reset to restore the SIV system to an operational state after each shut down event (N).
f. Be provided with thermal expansion relief to accommodate expansion of the liquid when closed. Thermal relief piping shall be properly sized and routed around the SIV, into the downstream segment of the pipeline or into other containment (N/E).

SIVs installed in liquid hazard class, H_C pipelines, or at a MOT with a seismic classification Medium or High per Table 4-1, shall be equipped with “Local” and “Remote” actuation capabilities. Local control SIVs may be motorized and/or operated manually (N).

8.4 FIRE DETECTION

A MOT shall have a permanently installed automated fire detection or sensing system (N).

8.5 FIRE ALARMS

Automatic and manual fire alarms shall be provided at strategic locations. The fire alarm system shall be arranged to provide a visual and audible alarm that can be readily discerned by all personnel at the MOT. Additionally, a visual alarm shall be displayed at the Facility’s Control Center (N/E).

If the fire alarm system is integrated with the ESD system, the operation shall be coordinated with the closure of SIVs, block valves and pumps to avoid adverse hydraulic conditions (N/E).

8.6 FIRE SUPPRESSION

Table 8-3 gives the minimum provisions for fire water flow rates and fire extinguishers. The table includes consideration of the fire hazard classification (Low, Medium or High), the cargo liquid hazard class (Low or High) and the vessel or barge size. The minimum provisions may have to be augmented for multi-berth terminals or those conducting simultaneous transfers, in accordance with the risks identified in the Fire Plan.

8.6.1 Coverage (N/E)

The fire suppression system shall provide coverage for:

a. Marine structures including pier/ wharf, approach trestle
b. Terminal cargo manifold
c. Cargo transfer system including loading arms, hoses and hose racks
d. Vessel manifold
e. Sumps
f. Pipelines
g. Control Stations

8.6.2 Fire Hydrants

Hydrants shall be located not greater than 300 ft. apart, along the wharf and approach trestle. Hose connections shall be provided at the base of fixed monitors and upstream of the water and foam isolation valves. The system shall be accessible to fire trucks or mutual aid equipment as identified in the Fire Plan (N).

Hydrants and hoses shall be capable of applying two independent water streams covering the cargo manifold, transfer system, vessel manifold and sumps (N/E).

8.6.3 Fire Water

The source of fire water should be reliable and provide sufficient capacity as determined in the fire plan.

a. All wet systems shall be maintained pressurized (jockey pump or other means) (N/E).
b. Wet system headers shall be equipped with a low-pressure alarm wired to the control room (N).

c. Fire pumps shall be installed at a distance of at least 100 ft. from the nearest cargo manifold area (N).

d. Hose connections for fireboats or tugboats shall be provided on the MOT fire water line. Connections shall be installed at a safe access distance from the high-risk areas such as sump, manifold and loading arms (N/E).

8.6.4 Foam Supply (N/E)

Product flammability, foam type, water flow rates and application duration shall be considered in foam supply calculations.

Fixed foam proportioning equipment shall be located at a distance of at least 100 ft. from the high-risk areas such as sump, manifold and loading arms, except where hydraulic limits of the foam delivery system require closer proximity.

8.6.5 Water/Foam Monitor Systems

Fire monitors (water/foam) shall be located to provide coverage of cargo manifolds, loading arms, hoses, and vessel manifold areas. This coverage shall provide at least two independent streams of water/foam. Monitors shall be located to provide an unobstructed path between the monitor and the target area (N/E).

In cases where the vessel's maximum manifold elevation is less than 30 ft. above the loading platform, the monitors may be mounted on the wharf deck. If the vessel manifold is more than 30 ft. above the platform, the following factors shall be considered, to determine if elevated masts or towers are required (N/E):

- Maximum tanker freeboard
- Tidal variations
- Pier/wharf/loading platform elevation
- Winds
- Fire water line pressure

Sprinklers and/or remotely controlled water/foam monitors shall be installed to protect personnel, escape routes, shelter locations and the fire water system (N).

Isolation valves shall be installed in the fire water and the foam lines in order to segregate damaged sections without disabling the entire system. Readily accessible isolation valves shall be installed 100 – 150 ft from the manifold and the loading arm/hose area (N).

8.6.6 Supplemental Fire Suppression Systems (E)

A supplemental system is an external waterborne or land-based source providing water and/or foam for fire suppression. Supplemental systems may <u>not</u> provide more than one-quarter of the total water/foam requirements specified in the Fire Plan and Table 8-3.

Additionally, supplementary systems shall not be considered in a Fire Plan, unless available within 20 minutes following the initiation of a fire alarm. Mutual aid may be considered as part of the supplemental system.

8.7 REFERENCES

[8.1] 49 CFR 195.452 "Pipeline Integrity Management in High Consequence Areas"

[8.2] Department of Defense, 2000, MIL-STD-882D, "Standard Practice for System Safety," Washington, D.C.

TABLE 8-3

MINIMUM FIRE SUPPRESSION PROVISIONS (N/E)

Fire Hazard Classification (From Table 8-2)	Vessel and Cargo Liquid Hazard Class (From Table 8-1)	MINIMUM PROVISIONS
LOW	Barge with L_C (including drums)	**500 gpm of water** 2 x 20 lb. portable dry chemical or 2 x 110 lb. wheeled dry chemical extinguishers or the equivalent.
LOW	Barge with H_C (including drums) Tankers < 50 KDWT, handling L_C or H_C	**1,500 gpm of water** 2 x 20 lb. portable dry chemical or 2 x 165 lb. wheeled dry chemical extinguishers or the equivalent.
MEDIUM	Tankers < 50 KDWT handling L_C	**1,500 gpm of water** 2 x 20 lb. portable dry chemical or 2 x 165 lb. wheeled dry chemical extinguishers or the equivalent.
MEDIUM	Tankers < 50 KDWT, handling H_C	**2,000 gpm of water** 4 x 20 lb. portable dry chemical or 2 x 165 lb. wheeled dry chemical extinguishers or the equivalent.
HIGH	Tankers < 50 KDWT, handling L_C or H_C	**3,000 gpm of water** 4 x 20 lb. portable dry chemical or 2 x 165 lb. wheeled dry chemical extinguishers or the equivalent. .
LOW, MEDIUM, HIGH	Tankers > 50 KDWT, handling L_C or H_C	**3,000 gpm of water** 6 x 20 lb. portable dry chemical or 4 x 110 lb. wheeled dry chemical extinguishers or the equivalent.

Notes: L_C and H_C are defined in Table 8-1.
KDWT = Dead Weight Tons (Thousands)

9. PIPING AND PIPELINES

9.1 GENERAL

9.1.1 Purpose

Section 9 provides minimum engineering standards for piping, pipelines, supports and related appurtenances at Marine Oil Terminals (MOTs).

9.1.2 Applicability

Section 9 applies to new (N) and existing (E) piping and pipelines used for transferring:

- Oil (See Section 1.1) to or from tank vessels or barges
- Oil within the MOT
- Vapors, including Volatile Organic Compounds (VOCs)
- Inerting or enriching gases to vapor control systems

Additionally, Section 9 applies to all supports, valves, appurtenances, and for piping or pipelines providing services, including stripping, sampling, venting, vapor control, fire water.

The requirements for piping and pipelines shall apply to all existing MOTs and all in-kind replacement of short pipeline sections, piping, or minor modifications of components.

The requirements for piping and pipelines at new MOTs, and the addition of new (non-replacement) piping, pipelines, components or in-kind replacement of a major section of a pipeline and/or systems for existing MOTs, shall be considered as new.

9.1.3 Applicable Codes, Standards, and Recommended Practices

a. Regulations, Codes and Standards

Piping, pipeline systems and components shall conform to the requirements of Section 9, as well as the appropriate requirements of the following:

- 2 CCR Sections 2550 - 2556, 2560 - 2571
- 33 CFR Part 154, Subpart C - Equipment Requirements, and Subpart E – Vapor Control Systems
- 46 CFR Part 39 – Vapor Control Systems
- 49 CFR Part 195 – Transportation of Hazardous Liquids By Pipeline
- American Petroleum Institute (API), 2001, API Standard 1160, “Managing System Integrity for Hazardous Liquid Pipelines,” 1st ed., Washington, D.C.
- American Petroleum Institute (API), 1994, API Standard 2610, “Design, Construction, Operation, Maintenance, and Inspection of Terminal and Tank Facilities,” ANSI/API STD 2610-1994, 1st ed., Washington, D.C.
- American Petroleum Institute (API), 1996, API Standard 607, “Fire Test for Soft-Seated Quarter-Turn Valves,” 4th ed., 1993 (reaffirmed 4/1996), Washington, D.C.
- American Petroleum Institute (API), 1997, API Standard 609, “Butterfly Valves: Double Flanged, Lug- and Wafer-Type,” 5th ed., Washington, D.C.
- American Petroleum Institute (API), 1991, Recommended Practice 1124 (API RP 1124), “Ship, Barge, and Terminal Hydrocarbon Vapor Collection Manifolds,” 1st ed., Washington, D.C.
- American Society of Civil Engineers, 2000, “Minimum Design Loads for Buildings and Other Structures,” ASCE 7-98, Revision of ANSI/ASCE 7-95, Reston, VA.
- American Society of Mechanical Engineers (ASME), 1996, ASME B16.34, “Valves Flanged Threaded And Welding End,” New York.
- American Society of Mechanical Engineers (ASME), 1996, ASME B16.5,” Pipe Flanges and Flanged Fittings,” New York.
- American Society of Mechanical Engineers (ASME), 1999, ASME B31.3, “Process Piping,” New York.
- American Society of Mechanical Engineers (ASME), 1998, ASME B31.4, “Pipeline Transportation Systems For Liquid Hydrocarbons And Other Liquids,” New York.
- CalARP Program Seismic Guidance Committee, 1998, “Guidance for California Accidental Release Prevention (CalARP) Program Seismic Assessments”, Sacramento, CA.
- Federal Emergency Management Agency, Nov. 2000, FEMA -356, “Prestandard and Commentary for the Seismic Rehabilitation of Buildings”, FEMA 356, Washington, D.C.

- National Fire Protection Assoc., 2000, NFPA 30, “Flammable and Combustible Liquids Code,” Quincy, MA.

b. Recommended Practices

- International Chamber of Shipping (ICS), Oil Companies International Marine Forum (OCIMF), International Association of Ports and Harbors (IAPH), 1996, “International Safety Guide for Oil Tankers and Terminals (ISGOTT),” 4th ed., Witherby, London.
- Oil Companies International Marine Forum (OCIMF), 1987, “Guide on Marine Terminal Fire Protection and Emergency Evacuation,” 1st ed., Witherby, London.
- Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, 1997, “Guidelines for Seismic Evaluation and Design of Petrochemical Facilities”, American Society of Civil Engineers, New York.

9.2 OIL PIPING AND PIPE LINE SYSTEMS

All pressure piping and pipelines for oil service shall conform to the requirements of ASME B31.3 or B31.4 as appropriate, and API Standard 2610. Additional requirements are as follows:

a. All piping/pipelines shall be documented on current P&ID’s (N/E).

b. Piping and pipeline systems shall be installed above deck (N).

c. The systems shall be arranged in a way not to obstruct access to and removal of other piping components and equipment (N).

d. Flexibility shall be achieved through adequate expansion loops or joints (N/E).

e. A guide or lateral restraint shall be provided just past the elbow where a pipe changes direction in order to minimize excessive axial stress (N).

f. Piping shall be routed to allow for movement due to thermal expansion and seismic displacement, without exceeding the allowable stresses in the supports, and anchor connections (see Section 9.3) (N/E).

g. Plastic piping shall not be used unless designated for oil service (N/E).

h. If a flanged connection exists within 20 pipe diameters from the end of any replaced section, the pipe shall be replaced up to and including the flange (E).

i. Pipelines shall be seamless, electric-resistance-welded or electric-fusion-welded and conform to ASME B31.4. (N)

j. Piping greater than 2 inches in diameter shall be butt-welded. Piping 2 inches and smaller shall be socket welded or threaded.

k. Pipeline connections directly over the water shall be welded (N). Flanged connections not over water shall have secondary containment (N).

l. Existing pipelines that do not have a valid and certified Static Liquid Pressure Test (SLPT) (2 CCR 2567) shall be marked “OUT OF SERVICE”. Out-of-service piping/pipelines shall be purged, gas-freed and physically isolated from sources of oil (E).

m. If a pipeline is “out-of-service” for 3 or more years, it will require Division approval prior to re-use (E).

n. Where there are differing requirements between standards itemized herein, applications shall be selected based on design needs and system conditions, subject to Division approval.

9.3 PIPELINE STRESS ANALYSIS (N/E)

Pipeline stress analysis shall be performed for:

- New piping and pipelines
- Significant re-routing/relocation of existing piping
- Any replacement of “not in-kind” piping
- Any significant rearrangement or replacement of “not in-kind” anchors and/or supports
- Significant seismic displacements calculated from the structural assessment

Piping stress analysis shall be performed in accordance with ASME B31.4, considering all relevant loads and corresponding displacements determined from the structural analysis described in Section 4.

Flexibility analysis for piping, considering supports, shall be performed in accordance with ASME B31.4 by using the largest temperature differential imposed by normal operation, start-up, shutdown, or abnormal conditions. Thermal loads shall be based upon maximum and minimum local temperatures; heat traced piping shall use the maximum attainable temperature of the heat tracing system.

To determine forces at sliding surfaces, the coefficients of static friction shown in Table 9-1 shall be used (N/E).

TABLE 9-1

COEFFICIENTS OF STATIC FRICTION

Sliding Surface Materials	Coefficient of Static Friction
Teflon on Teflon	0.10
Plastic on Steel	0.35
Steel on Steel	0.40
Steel on Concrete	0.45
Steel on Timber	0.49

9.4 ANCHORS AND SUPPORTS

Anchors and supports shall conform to ASME B31.3 and 31.4, API Standard 2610, and the ASCE Guidelines for Seismic Evaluation and Design of Petrochemical Facilities (N).

For existing piping, pipelines and supports, the seismic assessment shall be performed in accordance with CalARP or FEMA 356, as appropriate (E).

9.5 APPURTENANCES

9.5.1 Valves and Fittings

Valves and fittings shall meet the following requirements:

- Located, positioned and conform to ASME B 31.4, API Standard 609, and ASME B16.34, as appropriate, based on their service (N).
- Conform to API Standard 2610 (N/E).
- Stems shall be oriented in a way not to pose a hazard in operation or maintenance (N/E).
- Non-ductile iron, cast iron, and low-melting temperature metals shall not be used in any hydrocarbon service (N/E).
- Double-block and bleed valves shall be used for manifold valves. (N/E).
- Isolation valves shall be fire-safe, in accordance with API Standard 607 (N).
- Swing check valves shall not be installed in vertical down-flow piping (N/E).
- Pressure relief valves shall be used in any closed piping system that has the possibility of being over pressurized due to temperature increase or surging (thermal relief valves) (N/E).
- Any piping with blocked sections containing stagnant oil shall have a relief valve in the event of over pressurization due to temperature increase. Discharge from pressure relief valves shall be directed into lower pressure piping for recycling or proper disposal. Discharge shall never be directed into the open environment, unless secondary containment is provided (N/E).
- Threaded, socket-welded, flanged and welded fittings shall conform to API Standard 2610 (N/E).

9.5.2 Valve Actuators (N/E)

- Actuators shall have a readily-accessible, manually-operated overriding device to operate the valve during a power loss.
- Torque switches shall be set to stop the motor closing operation at a specified torque setting
- Limit switches shall be set to stop the motor opening operation at a specified limit switch setting.
- Critical valves shall be provided with thermal insulation. The insulation shall be inspected and maintained at periodic intervals. Records of thermal insulation inspections and condition shall be maintained for at least 6 years.
- Electrical insulation for critical valves shall be measured for resistance following installation and re-tested periodically. These records shall be maintained for at least 6 years.

9.6 UTILITY AND AUXILIARY PIPING SYSTEMS

Utility and auxiliary piping includes service for:

- Stripping and sampling
- Vapor control
- Fire water and foam
- Natural gas
- Compressed air, venting and nitrogen

Stripping and sampling piping shall conform to Section 9.2 and ANSI/ASME B31.3 (N/E).

Vapor return lines and VOC vapor inerting and enriching/incinerating (natural gas) piping shall conform to 33 CFR Part 154, Subpart E, ANSI/ASME B31.3 and API RP 1124 (N).

Fire water and foam piping shall meet the following requirements:

- Conform to ASME B 16.5
- Fire mains shall be carbon steel pipe (N/E)
- High density polyethylene (HDPE) piping may be used for buried pipelines (N/E)
- Piping shall be color-coded (N/E)

Compressed air, venting and nitrogen piping shall conform to ASME B31.3 (N).

10. MECHANICAL AND ELECTRICAL EQUIPMENT

10.1 GENERAL

10.1.1 Purpose

Section 10 provides the minimum standards for mechanical and electrical equipment at MOTs.

10.1.2 Applicability

Section 10 applies to all new (N) and existing (E) MOTs.

The addition of new (non-replacement) equipment, components and systems installed after these regulations become effective shall conform to the 'New' (N) requirements of Section 10. Work directly associated with these installations or modifications shall also comply with these requirements.

Replacement of in-kind existing equipment or associated components and minor modifications of components shall meet 'Existing' (E) provisions.

10.2 MARINE LOADING ARMS

10.2.1 Applicable Codes, Standards and Recommended Practices

Marine loading arms and ancillary systems shall conform to the requirements of Section 10.2 as well as the appropriate requirements of the following regulations, codes, standards guidelines and recommended practices:

2 CCR 2370 Communications

2 CCR 2380 (b) Loading Arms

33 CFR 54.510 Loading Arms

47 CFR 90 Private Land Mobile Radio Services

American Society of Mechanical Engineers (ASME), 2000, ASME B40.100-1998, "Pressure Gauges and Gauge Attachments," New York.

International Chamber of Shipping (ICS), Oil Companies International Marine Forum (OCIMF), International Association of Ports and Harbors (IAPH), 1996, "International Safety Guide for Oil Tankers and Terminals (ISGOTT)," 4th ed., Witherby, London.

National Fire Protection Association, 2002, NFPA 70, Articles 430, 500, 501 and 504, "National Electric Code," Quincy, MA.

National Fluid Power Association (NFPA), 1996, ANSI/(NFPA) T3.6.7 R2-1996, "Fluid Power Systems and Products – Square Head Industrial Cylinders – Mounting Dimensions," Milwaukee, WI.

Oil Companies International Marine Forum (OCIMF), 1999, "Design and Construction Specification for Marine Loading Arms," 3rd ed., Witherby, London.

Task Committee on Seismic Evaluation and Design of Petrochemical Facilities 1997, "Guidelines for Seismic Evaluation and Design of Petrochemical Facilities", American Society of Civil Engineers, New York.

Underwriters Laboratory, Inc., 1997, "Intrinsically Safe Apparatus and Associated Apparatus for Use in Class I, II, III, Division 1, Hazardous (Classified) Locations," ANSI/UL Standard No. 913, 5th ed., Northbrook, IL.

10.2.2 General Criteria

The maximum allowable extension limits shall consider the following:

- Vessel sizes and manifold locations
- Lowest-low water level (Datum)
- Highest-high water level
- Maximum vessel surge and sway
- Maximum width of fendering system

The loading arm, tie-downs and components shall meet site-specific seismic criteria (N).

10.2.3 Electrical and Hydraulic Power Systems

a. Pressure and Control Systems (N)

- Pressure gauges shall be mounted in accordance with the requirements of ASME B40.100-1998. The hydraulic drive cylinders shall be mounted and meet either the mounting requirements of ANSI/(NFPA) T3.6.7 R2-1996 or equivalent.

- The control system, quick disconnect couplings, and emergency release systems shall be in conformance with the provisions of the OCIMF (see 10.2.1).
- Out-of-limit, balance and the approach of out-of-limit alarms shall be located at or near the loading arm console.

b. Electrical Components (N)

All electrical equipment, wiring, cables, controls and electrical auxiliaries located in hazardous areas shall comply with Section 11. In addition, the following criteria shall be implemented.

- Equipment shall be provided with a safety disconnecting device to isolate the entire electrical system from the electrical mains in accordance with NEC, Article 430.
- Motor controllers and 3-pole motor overload protection shall be located and sized in accordance with NEC, Article 430.
- Control circuits shall be limited to 120 volts and shall comply with NEC, Articles 500 and 501. Alternatively, intrinsically safe wiring and controls may be provided in accordance with NEC Article 504 and ANSI/UL Std No. 913.
- Grounding and bonding shall comply with the requirements in Section 11 and NEC Article 430.

c. Remote Operation

The remote control system, where provided, shall conform to the recommendations of the OCIMF (see 10.2.1). The remote operation shall be facilitated by either a pendant control system or by a hand-held radio controller (N).

The pendant control system shall be equipped with a plug-in capability to an active connector located either in the vicinity of the loading arms, or at the loading arm outboard end on the triple swivel, and hard-wired into the control console. The umbilical cord running from the triple swivel to the control console shall be attached to the loading arm. Other umbilical cords shall have sufficient length to reach the maximum envelope (N).

The radio controller if installed shall comply with 2 CCR 2370 and Federal Communications Commission (47 CFR Part 90) requirements for transmitters operating in an industrial environment (N/E).

10.3 OIL TRANSFER HOSES

10.3.1 Applicable Codes, Standards, and Recommended Practices

Oil transfer hoses shall conform to the requirements of Section 10.3, as well as the appropriate requirements of the following:

2 CCR 2380 (a) Hose Assemblies

33 CFR 154.500 and 154.520

American Society for Testing and Materials, 2001, ASTM F-1122-87 (1998), "Standard Specification for Quick Disconnect Couplings," West Conshohocken, PA.

American Society of Mechanical Engineers, 1996, ASME/ANSI B16.5, "Pipe Flanges and Flanged Fittings," New York

American Petroleum Institute (API), 1994, API Standard 2610, "Design, Construction, Operation, Maintenance, and Inspection of Terminal and Tank Facilities," ANSI/API STD 2610-1994, 1st ed., Washington, D.C.

International Chamber of Shipping (ICS), Oil Companies International Marine Forum (OCIMF), International Association of Ports and Harbors (IAPH), 1996, "International Safety Guide for Oil Tankers and Terminals (ISGOTT)," 4th ed., Witherby, London.

10.3.2 General Criteria (N/E)

- Cargo transfer hoses for oil transfer service shall comply with 2 CCR 2380, 33 CFR 154.500 and ISGOTT (see 10.3.1). Hoses with diameters of 6 inches or larger shall have flanges that meet ANSI B16.5 (see 10.3.1). Hoses with diameters of 4 inches or less may have quick disconnect fittings provided that they meet ASTM F-1122 (see 10.3.1).

10.4 LIFTING EQUIPMENT: WINCHES AND CRANES

10.4.1 Applicable Regulations, Codes and Standards

Lifting equipment shall conform to the requirements of Section 10.4, as well as the appropriate requirements of the following:

29 CFR Parts 1910, 1917, 1918.

American Society of Mechanical Engineers, 1996, ASME B30.4 - 1996, "Portal Tower and Pedestal Cranes," New York.

American Society of Mechanical Engineers, 2002, ASME B30.7 - 2001, "Base Mounted Drum Hoists," New York.

American Society of Mechanical Engineers, 1999, ASME HST-4, "Performance Standard for Overhead Electric Wire-Rope Hoists," New York.

10.4.2 Winches

In addition to the applicable regulations, standards and codes specified in Section 10.4.1, the following requirements shall apply:

- Winches and ancillary equipment shall be suitable for a marine environment (N/E).
- Winches shall be provided with a fail-safe braking system, capable of holding the load under all conditions, including a power failure (N/E).
- Winch drums shall comply with ASME B30.7 (N).
- Winches shall be fully reversible (N).
- Shock, transient, and abnormal loads shall be considered when selecting winch systems (N).
- Winches shall have limit switches and automatic trip devices to prevent over-travel of the drum in either direction. Limit switches shall be tested, and demonstrated to function correctly under all operating conditions without inducing undue tensions or slack in the winch cables (N/E).
- Electrical equipment shall conform to the provisions of Section 11.
- Under all operating conditions, there shall be at least two full turns of cable on grooved drums, and at least three full turns on ungrooved drums (N/E).

10.4.3 Cranes (N/E)

In addition to the applicable regulations, standards and codes specified in Section 10.4.1, the following requirements shall apply:

- Cranes shall not be loaded in excess of the manufacturer's rating except during performance tests.
- Drums on load-hoisting equipment shall be equipped with positive holding devices. Under all operating conditions, there shall be at least two full turns of cable on grooved drums, and at least three full turns on ungrooved drums.
- Braking equipment capable of stopping, lowering, and holding a load of at least the full test load shall be provided.
- When not in use, crane booms shall be lowered to ground level or secured to a rest support against displacement by wind loads or other outside forces.
- Safety systems including devices that affect the safe lifting and handling, such as interlocks, limit switches, load/moment and overload indicators with shutdown capability, emergency stop switches, radius and locking indicators, shall be provided.

10.5 SHORE-TO-VESSEL ACCESS FOR PERSONNEL

10.5.1 Applicability

Section 10.5 applies to shore-to-vessel means of access for personnel and equipment provided by the terminal. This includes ancillary structures and equipment, which support, supplement, deploy and maneuver such vessel access systems.

10.5.2 Applicable Codes, Standards, and Recommended Practices

Shore-to-vessel access for personnel shall conform to the requirements of Section 10.5, as well as the appropriate requirements of the following:

29 CFR 1910, 1915, 1917, and 1918

US Army Corps of Engineers, 1996, “Safety and Health Requirements Manual, Sections 19 and 21,” EM 385-1-1, Washington, D.C.,

National Fire Protection Association, 2002, NFPA 70, “National Electric Code,” Quincy, MA.

10.5.3 General Criteria

- Shore-to-vessel access systems shall be designed to withstand the forces from dead, live, wind, vibration, impact loads, and the appropriate combination of these loads. The design shall consider all the critical positions of the system in the stored, maintenance, maneuvering, and deployed positions, where applicable (N).
- The minimum live load shall be 50 psf and 25 psf for all handrails (N/E).
- The walkway shall be not less than 36 inches in width (N) and not less than 20 inches for existing walkways (E).
- The shore-to-vessel access system shall be positioned so as to not interfere with the safe passage or evacuation of personnel (N/E).
- Electrical and instrumentation components shall comply with NFPA 70 (see 10.5.2).
- Guardrails shall be provided on both sides of the access systems with a clearance between the inner most surfaces of the guardrails of not less than 36 inches and shall be maintained the full length of the walkway (N).
- Guardrails shall be at a height not less than 33 inches above the walkway surface and shall include an intermediate rail located midway between the walkway surface and the top rail (N/E).
- The walkway surface, including self-leveling treads, if so equipped, shall be finished with a safe non-slip footing accommodating all operating gangway inclinations (N/E).
- In meeting the requirements of the operating envelope, under no circumstances shall the operating inclination of the walkway exceed 60 degrees or the maximum angle recommended by the manufacturer, whichever is less, either above or below the horizontal (N/E).

10.6 SUMPS, DISCHARGE CONTAINMENT AND ANCILLARY EQUIPMENT

10.6.1 Applicable Regulations and Recommended Practice

Sumps, discharge containment and ancillary equipment shall conform to the requirements of Section 10.6, as well as the appropriate requirements of the following:

- 2 CCR 2380 (f)
- 33 CFR 154.530 - Small Discharge Containment
- 40 CFR 112 - Oil Pollution Prevention
- American Petroleum Institute, 1991, API Recommended Practice 1125 (API RP 1125), “Overfill Control Systems for Tank Barges,” 1st ed., Washington, D.C.

10.6.2 General Criteria

- Sumps for oil drainage shall be equipped with pressure/vacuum vents, automatic draining pumps and shall be tightly covered (N/E).
- Sumps which provide drainage for more than one berth should be equipped with liquid seals so that a fire on one berth does not spread (N/E) via the sump.
- Sumps shall be located at least 25 ft. from the manifolds, base of the loading arms or hose towers (N).

10.7 VAPOR CONTROL SYSTEMS

Vapor control systems shall conform to the appropriate requirements of the following:

- 2 CCR 2550 through 2556
- 33 CFR 154.800 through 154.850
- American Petroleum Institute, 1991, API Recommended Practice 1124 (API RP 1124), “Ship, Barge, and Terminal Hydrocarbon Vapor Collection Manifolds,” 1st ed., Washington, D.C.
- American Petroleum Institute, 1994, API Standard 2610 (ANSI/API STD 2610-1994), “Design, Construction, Operation, Maintenance, and Inspection of Terminal and Tank Facilities,” 1st ed., Washington, D.C.

10.8 EQUIPMENT ANCHORS AND SUPPORTS

Anchors and supports of electrical and mechanical equipment including seismic assessment, (N) shall conform to:

- American Petroleum Institute, 1994, API Standard 2610 (ANSI/API STD 2610-1994), "Design, Construction, Operation, Maintenance, and Inspection of Terminal and Tank Facilities," 1st ed., Washington, D.C.
- American Society of Mechanical Engineers (ASME), 1999, ASME B31.3, "Process Piping," New York.
- American Society of Mechanical Engineers (ASME), 1998, ASME B31.4, "Pipeline Transportation Systems For Liquid Hydrocarbons And Other Liquids," New York.
- Task Committee on Seismic Evaluation and Design of Petrochemical Facilities 1997, "Guidelines for Seismic Evaluation and Design of Petrochemical Facilities", American Society of Civil Engineers, New York.

For existing (E) equipment, the seismic assessment (see 4.6) shall be performed in accordance with CalARP or FEMA 356, as appropriate.

- CalARP Program Seismic Guidance Committee, 1998, "Guidance for California Accidental Release Prevention (CalARP) Program Seismic Assessments," Sacramento, CA.
- Federal Emergency Management Agency, 2000, FEMA-356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Washington, D.C.

11. ELECTRICAL SYSTEMS

11.1 GENERAL

Electrical systems include the incoming electrical service and components, the electrical distribution system, branch circuit cables and the connections. Also included are:

- Lighting, for operations, security and navigation
- Controls for mechanical and electrical equipment
- Supervision and instrumentation systems for mechanical and electrical equipment
- Grounding and bonding
- Corrosion protection through cathodic protection
- Communications and data handling
- Fire detection
- Fire alarm systems
- Emergency shutdown systems (ESD)

11.1.1 Purpose

Section 11 provides minimum standards for electrical systems at MOTs.

11.1.2 Applicability

a. New Marine Oil Terminals (N)

The new (N) requirements of Section 11 shall be applied to all new terminals or existing terminals which recommence operations with a new, approved operations manual (2 CCR 2385).

b. Existing Marine Oil Terminals (N/E)

The new (N) requirements of Section 11 shall be applied to the installation of new equipment, new systems, repairs or substantially modified in-place systems. Work associated with these installations, repairs or modifications shall comply with the new (N) requirements of Section 11.

Existing (E) requirements apply to all other electrical systems and components.

11.1.3 Applicable Codes, Standards, and Recommended Practices

a. Regulations, Codes and Standards

Electrical equipment and ancillary systems shall conform to the requirements of Section 11, as well as the appropriate requirements of the following:

- 2 CCR 2341, 2365, 2370 and 2380(h)
- 29 CFR 1917.123 (a) - Illumination
- 33 CFR 154.570 (d) - Lighting
- 33 CFR 154.812 - Facility Requirements for Vessel Liquid Overfill Protection
- 40 CFR 112.7 (e)(2) Bulk Storage Tanks (onshore); (excluding production facilities)
- American Petroleum Institute, 1994, API Standard 2610 (ANSI/API STD 2610-1994), "Design, Construction, Operation, Maintenance, and Inspection of Terminal and Tank Facilities," 1st ed., Washington, D.C.
- American Petroleum Institute, 1996, API Recommended Practice 2350 (API RP 2350), "Overfill Protection for Storage Tanks," 2nd ed., Washington, D.C.
- American Petroleum Institute, 1995, API Recommended Practice 1125 (API RP 1125), "Overfill Control Systems for Tank Barges," 1st ed., February 1991 (Reaffirmed 1995), Washington, D.C.
- American Petroleum Institute, 2002, API 570, "Piping Inspection Code", 2nd ed., October 1998 (February 2000 Addendum 1), Washington, D.C.
- American Petroleum Institute, 1999, API Recommended Practice 540 (API RP 540), "Electrical Installations in Petroleum Processing Plants," 4th ed., Washington, D.C.
- American Petroleum Institute, 1997, API Recommended Practice 505 (API RP 505), "Recommended Practice for Classification of Locations for Electrical Installations at Petroleum Facilities Classified as Class I, Zone 0, Zone 1 and Zone 2," 1st ed., Washington, D.C.
- American Petroleum Institute, 1997, API Recommended Practice 500 (API RP 500), "Recommended Practice for Classification of Locations for Electrical Installations at Petroleum Facilities Classified as Class I, Division 1 and Division 2," 2nd ed., Washington, D.C.

- National Fire Protection Association, 2002, NFPA 70, "National Electric Code (NEC)," Quincy, MA.
- National Fire Protection Association, 1998, NFPA 496, "Standard for Purged and Pressurized Enclosures for Electrical Equipment," Quincy, MA.

b. Recommended Practices

- American Petroleum Institute, 1998, API Recommended Practice 2003 (API RP 2003), "Protection Against Ignitions Arising out of Static, Lightning and Stray Currents," 6th ed., Washington, D.C.
- International Chamber of Shipping (ICS), Oil Companies International Marine Forum (OCIMF), International Association of Ports and Harbors (IAPH), 1996, "International Safety Guide for Oil Tankers and Terminals (ISGOTT)," 4th ed., Witherby, London.
- National Association of Corrosion Engineers (NACE), Standard Recommended Practice, 1994, RP0176-1994 "Corrosion Control of Steel Fixed Offshore Platforms Associated with Petroleum Production," Houston, TX.
- Oil Companies International Marine Forum (OCIMF), 1987, "Guide on Marine Terminal Fire Protection and Emergency Evacuation," 1st ed., Witherby, London.

11.2 HAZARDOUS AREA DESIGNATIONS AND PLANS (N/E)

Area classifications shall be determined in accordance with the requirements of API RP 500, using NEC, Chapter 5 (Articles 500, 501, 504, 505, 510 and 515 as applicable). A marine oil terminal shall have a current set of scaled plan drawings, with clearly designated areas showing the hazard class, division and group. The plan view shall be supplemented with sections, elevations and details to clearly delineate the area classification at all elevations starting from low water level. The drawings shall be certified by a Professional Electrical Engineer. The plans shall be reviewed, and revised when modifications to the structure, product or equipment change hazardous area identifications or boundaries.

11.3 IDENTIFICATION AND TAGGING

All electrical equipment, cables, conductors shall be clearly identified by means of tags, plates, color coding or other approved means to facilitate troubleshooting and improve safety, and shall conform to the identification carried out for the adjacent on-shore facilities (N). Articles and topics applicable for such identification are found in the NEC.

Existing electrical equipment (E) shall be tagged.

Where identification is necessary for the proper and safe operation of the equipment, the marking shall be clearly visible and illuminated (N/E). A coded identification system shall apply to all circuits, carrying low or high voltage power, control, supervisory or communication (N).

11.4 PURGED OR PRESSURIZED EQUIPMENT IN HAZARDOUS LOCATIONS (N/E)

Purged or pressurized enclosures shall be capable of preventing the entry of combustible gases into such spaces, in accordance with NFPA – 496. Special emphasis shall be placed on reliability and ease of operation. The pressurizing equipment shall be electrically monitored and alarms shall be provided to indicate failure of the pressurizing or purging systems.

11.5 ELECTRICAL SERVICE

The capacity of the electrical service feeders and transformers shall be adequate to serve the peak demand of all electrical loads (N/E).

Where critical circuits are used for spill prevention, fire control or life safety, an alternative service derived from a separate source and conduit system, shall be located at a safe distance from the main power service. A separate feeder from a double-ended substation or other source backed up by emergency generators will meet this requirement. An uninterrupted power service (UPS) shall be provided for control and supervisory circuits associated with ESD systems (N).

Emergency cables and conductors supplying block valve motors, starters and associated conduits and conductors shall be fire rated and remain in service for

15 minutes in a 2000° F fire. The temperature around the critical components shall not exceed 200° F during the 15 minutes (N).

Wiring in fireproofed conduits shall be derated 15% to account for heat buildup during normal operation. Type MI cables may be used in lieu of fireproofing of wiring (N).

a. Emergency cables and conductors shall be located where they are protected from damage caused by traffic, corrosion or other sources (N).
b. Allowance shall be made for electrical faults, overvoltages and other abnormalities (N).
c. Where solid state motor controls are used for starting and speed control, corrective measures shall be incorporated for mitigating the possible generation of harmonic currents that may affect the ESD or other critical systems (N).

11.6 GROUNDING AND BONDING (N/E)

a. All electrical equipment shall be effectively grounded by means of an approved electrode or other means in accordance with NEC Article 250. All non-current carrying metallic equipment, structures, piping and other elements shall be effectively bonded to the grounded system.
b. Grounding shall consider any active corrosion protection system for on-shore piping, submerged support structures or other systems. Insulation barriers, including flanges or non-conducting hoses shall be used to isolate cathodic protection systems from other electrical/static sources. None of these systems shall be compromised by grounding or bonding arrangements that may interconnect the corrosion protection systems or interfere with them in any way that would reduce their effectiveness. Bonding shall be applied to all metallic components.
c. Bonding of vessels to the MOT structure is not permitted (2 CCR 2341 (f)).
d. Whenever flanges of pipelines with cathodic protection are to be opened for repair or other work, the flanges shall be bonded prior to separation.
e. Direct wiring to ground shall be provided from all towers, loading arms or other high structures that are susceptible to lightning surges or strikes.

11.7 EQUIPMENT SPECIFICATIONS (N)

All electrical systems and components shall be certified by the National Electrical Manufacturers Association (NEMA) or the Nationally Recognized Testing Laboratories (NRTL). The equipment label and certification shall state that the item has been tested in accordance with the specified organization's test methods and that the item complies with the specified organization's reference standard.

11.8 ILLUMINATION (N/E)

Lighting shall conform to 2 CCR Section 2365 and 33 CFR 154.570 (d).

11.9 COMMUNICATIONS AND CONTROL SYSTEMS

11.9.1 Communication Systems (N/E)

Communications systems shall comply with 2 CCR 2370, and also incorporate appropriate recommendations contained in the OCIMF "Guide on Marine Terminal Fire Protection and Emergency Evacuation" (see 11.1.3).

11.9.2 Overfill Monitoring and Controls (N/E)

Overfill protection systems shall meet the provisions of API RP 2350 and 40 CFR 112.7. These systems shall be tested before each transfer operation or monthly, whichever is less frequent. Where vessel or barge overfill sensors and alarms are provided, they shall comply with 33 CFR154.812.

All sumps shall be provided with level sensing devices to initiate an alarm to alert the operator at the approach of a high level condition. A second alarm shall be initiated at a high-high level point to alert the operator. Unless gravity drainage is provided, sumps must have an automatic pump, programmed to start at a pre-determined safe level.

11.10 CORROSION PROTECTION

11.10.1 Corrosion Assessment (N)

A corrosion assessment shall be performed to determine environmental corrosivity. This assessment should include all steel or metallic components, including the structure, pipelines, supports or other ancillary equipment, with drawings and specifications for corrosion prevention/protection. The assessment shall be performed by a licensed professional engineer, using the methods and criteria prescribed by the National Association of Corrosion Engineers (NACE).

11.10.2 Inspection, Testing and Records (N/E)

For sacrificial anode systems, periodic underwater inspections shall be performed and observations recorded. For impressed current systems, monthly rectifier readings and annual potential readings of the protected components shall be taken. If the readings are outside of the acceptable potential limits, corrective actions shall be taken.

All isolating sections shall be tested immediately after installation or replacement, and, at a minimum, annually and recorded. Electrical tests on insulating flanges shall make use of specialized insulator testers. The test instrument shall make use of RF signals, capacitive measurements or other means to clearly determine whether an insulating flange is shorted or open circuited without being affected by pipe-to-soil potentials, cathodic protection voltages or whether it is buried or exposed.

The cathodic protection for buried pipelines shall conform to API 570.

Insulating and isolating arrangements for protection against static, stray and impressed currents shall be tested in accordance with 2 CCR 2341 and 2380.